AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

Paper No. 931.

SEPARATE *VERSUS* GENERAL CONTRACTS.

An Informal Discussion at the Annual Convention, May 21st, 1902.

SUBJECT FOR DISCUSSION:

"In contract work, either public or private, is it preferable to make separate contracts for the different branches of trades involved, or to combine all under one general contract?"

By Messrs. GEORGE E. GIFFORD, T. CHALKLEY HATTON, CHARLES G. DARRACH, S. BENT RUSSELL, M. WARD EASBY, CHARLES WORTHINGTON and HORACE ANDREWS.

GEORGE E. GIFFORD, M. Am. Soc. C. E.—The topic under discussion will be regarded by the speaker primarily from the standpoint of the contractor, and by giving the views of the contractor, he hopes to show the owner, the purchaser, or the consulting engineer, that his own interests lie with the contractor in this matter. He believes that contractors almost universally will agree that the dividing up of contract work, so that each principal branch is under original contract with the purchaser, is preferable to the making of one general contract which must be sublet to specialists in the different trades. Mr. Gifford.

The speaker has talked with many contractors on this subject, and does not recollect ever hearing one say that he preferred the general contract system. It is recognized, of course, that there have been formed large contracting firms and corporations who are ready to undertake work of almost any magnitude, and involving many differ-

Mr. Gifford. ent trades. This has been brought about not so much from preference as from necessity, because the greater part of government and municipal work is being let in this way, and, in order to bid upon it at all, the contractor must take it as a whole. This is also true of some private work, especially the large building constructions.

In many cases the contracting firm consists simply of an office; practically all its actual operations are performed by sub-contractors, and the most important man in the whole outfit is the purchasing agent. He does what the consulting engineer or the architect should do at first hand for his clients. In other cases the contracting firm makes a specialty of some particular branch of work, we will say mason-work or steelwork, and actually carries out this portion of the contract with his own appliances or at his own shops. The remainder, which he cannot do himself, he is obliged to sublet, or "farm out," as it is called, or else undertake operations with which he is not familiar and which he can only carry out at a disadvantage. He would much prefer to have only such work as he himself can perform, but is obliged to take the remainder in order to get his portion. The consequence is he is loaded up with a lot of work which he is not adapted to perform, but for which he has to be financially responsible, and he either does it at a loss to himself or at a loss to the purchaser.

It may be urged that the proper way to overcome this difficulty is by a combination of contractors in different lines who are willing to unite their bids in a grand total, including only one profit, and each one then perform his portion of the work. This is no doubt what the purchaser, the engineer, or the architect thinks will be done when he receives proposals in this way. The only trouble with this theory is that it does not harmonize with practice. There is not an entire unanimity of opinion and complete trust in one another among contractors in different trades, which is no doubt to be regretted, but which fact, nevertheless, remains. In a community-of-interest contract someone has to be the principal; someone must be designated as the main contractor, must receive the instructions, confer with the owner or his agents, draw the pay and settle with the others. There is the rub.

What is the practical way in which general contract work is actually taken?

We will assume our contract to be a large city bridge, involving substructure, possibly with pneumatic foundations, dredging, cut-stone masonry, concrete, excavation, embankment, paving, sidewalks, perhaps some sewers to be extended or moved, and water pipes to be changed. Then we have the superstructure with a draw span which will involve not only the structural steel, but various kinds of machinery, electric motors, electric lighting plant, a little carpenter work, some automatic gates, fancy hand railing, asphalt paving, and painting. There have been bridges let in New York City the bidding sheet of which showed more than one hundred items for which prices had to be inserted.

We will assume further that the substructure and the superstructure (it being unnecessary to designate specifically before this Society just what items are included under the above heads) are approximately equal. The bridge builder, for example, begins to prepare his bid and finds that he must either combine with some other contractor who will take care of all parts of the work which are technically known as substructure or general contract work; or he must get prices on the separate items from tradesmen of all branches; or he may assume that he knows enough about work of this kind to make his own figures for everything. Mr. Gifford.

The chances are that the first-mentioned alternative will be met with the statement: "We will make you figures for our part of the work provided you will agree to tie up with us." This may be the most satisfactory way in the end, provided the bid is successful; if not, there follow the usual charges, expressed or implied, that the bidder was "thrown down" by a bid too high, and if he is the successful bidder and finds that other figures, better than those given to him, have been submitted, he regrets his inability to deal with a cheaper party.

This plan also presupposes absolute confidence between the parties to the proposal, as to fair dealing all around, and this does not always follow. Many of the largest contractors and steel workers absolutely refuse to make prices to other contractors in advance of the receipt of the contract; and in some cases will even decline to bid to another contractor, even though he holds the contract; the position of sub-contractor being so undesirable, for many reasons, particularly the inability to deal directly with the owner, and the financial delays and losses involved in case the main contractor is slow pay or becomes financially embarrassed. The speaker knows what he is saying, in making these statements, from the fact that the firm which he represents almost invariably declines to assume the position of sub-contractor on any work. He knows of others who feel the same way.

Returning to the second method of preparing a proposal, that of getting separate prices on all the different classes of work, we find a marked disinclination, on the part of dealers and tradesmen, to give their best figures for bidding purposes, if they will give them at all. An inquiry of the sort is likely to be met with the question: "Have you this contract?" The answer being "No," it is likely that an excuse will be offered, such as "too busy to figure just now," or else the statement: "We will give you a safe price with which to bid; but, in case you secure the contract, we may be able to do better, and expect to hear from you again." Is this likely to be satisfactory to the bidder desiring to make a close bid? And after he had added his 5%, 10%, or whatever he sees fit to cover the financing of the job, contingencies, and his profit, is it likely to be an economical method for the purchaser?

Mr. Gifford. No doubt, if every contractor had the skill, and knowledge of costs and conditions to make up his own figures for all classes of work, it might be best to make a general contract. Sometimes he attempts to do this, with the result that he is entirely too liberal in his estimate, and loses the work, or else makes a mistake somewhere, and, when he comes to do the work or purchase his supplies, finds that there is a corner against him, or some condition has arisen in the market or in labor which he has not accounted for. It is practically impossible for one man or one firm to be fully posted on all the branches of work entering into a large bridge or building contract. The consequence is he finds that he has a losing contract, a highly undesirable state of affairs, not only for himself, but for the purchaser. There is probably no engineer or owner who desires to see his contractor lose money; it is certainly bad for the contractor, and a source of great trouble to the purchaser, if not an actual detriment to the work. Of course, it is not right to assume that all contracts will be profitable, even if taken and prosecuted by specialists; all are likely to make mistakes; but it is fair to presume that fewer mistakes are made in one's own line of work than in something with which one is not familiar.

The subject has thus far been considered principally from the standpoint of the contractor, and the speaker has endeavored to show wherein it would benefit the contractor to take only his own kind of work. In order to bring this about it must be shown to the purchaser, or the engineer or architect who represents him, that he will benefit by dealing separately. The principal argument offered in support of the general contract system is the supposed advantage of having only one party to deal with, and the presumption that trouble among independent contractors on the same work will be avoided by having them all under one control. The first proposition practically includes the second, and, while it may avoid some clerical or office work, it surely does not avoid anything in superintendence or inspection; and the fact that the combining of a large contract under one head will eliminate a large amount of competition would seem to far outweigh a slight saving of detail in drawing up a few extra contracts or making a few extra vouchers for payment.

It is a fact that the general-contract method eliminates competition. As an example, may be quoted the bridge work for the State of New York. The bulk of this work consists of not very large bridges over the canals, some of them aggregating perhaps $50 000, but most of them much less. It is the invariable practice in the Department of Public Works to make one contract to cover each structure, and they nearly all involve a little concrete, a little excavation, a little embankment, and so on, together with the steel superstructure, which, in the case of lift-bridges, may be quite complicated. The conse-

quence is that there is little or no competition in this State work, and in case they are so fortunate as to receive two or three bids, about half the time the bids exceed the appropriation, because excessive prices are put in for the little odds and ends of work which the bridge builder does not want, but has to take in order to get the steelwork. Mr. Gifford.

A local contractor, knowing or being able to ascertain easily the conditions prevailing, could take care of these items well and economically, but he is practically debarred from bidding because he cannot bid on the bridge and cannot find any bridge company who will give him a price. The bridge companies, on the other hand, at least a good many of them, not wishing to be bothered with the substructure work, especially under present conditions, when there is plenty of more desirable work, do not prepare any estimate, and the result is as stated; there is practically no competition and the prices are excessive.

There is no doubt whatever that much better results would be obtained if this work could be divided into, perhaps, not more than two separate contracts. The speaker once took occasion to ask an official of the State why they adhered to this practice, and found that it is because when the laws making the appropriations are passed, they direct that such and such moneys be appropriated and "a contract" be entered into for the work. The officials feel that they are bound to adhere rigidly to the letter of the law. It seems to be nobody's business to change this form of wording. This is only a typical case. The speaker could quote many instances where, of his own knowledge, the method of combining different kinds of work under one contract has prevented competition and raised the cost.

It seems strange that among the chief adherents to this method is the United States, in its various departments, and the largest municipalities of the country. We scarcely ever find it among the railroad companies, who regard the economical aspect more carefully than public officials. The Navy Department, for example, in letting the contract for a building, will include all masonry, carpentry, steelwork, plumbing, etc., under one contract. This surely precludes a certain amount of competition, and presumably is an expensive method, for the reasons stated above. Some of the principal bridge companies have practically turned down Navy Department work, although quite desirable in itself; but it cannot be secured without taking much work which is not desirable, or else under a sub-contract, with its attendant disagreeable features.

The same comment applies to the Treasury Department work, let by the Supervising Architect. Not long ago an officer of one of the large bridge companies wrote to the speaker, as follows:

"The National Association of Metal Work Manufacturers has been for months at work on the Government to get them to separate steel

Mr. Gifford. and ironwork from general contracts for public buildings, and believe they are about to succeed."

The speaker hopes they may succeed, but has not yet been informed that they have done so.

Within the last few months the speaker received the following inquiry, in a letter from the engineer of one of the largest bridge structures now in progress in this country:

"Do you prefer bidding on substructure and superstructure together, in one contract, or bidding on the superstructure alone? What are your controlling reasons therefor? It is assumed in this question that the cost of substructure is not less than one-third of the entire contract."

To which the speaker replied as follows:

"We prefer bidding on the superstructure alone, making the substructure contract an independent one, to be handled by other parties, presumably those in that special line of work. As to our reasons, the general statement that we are in the bridge business only, and understand it only, would seem to make any other reason unnecessary. The employment of sub-contractors is frequently a nuisance, and always undesirable to us."

The speaker was afterward informed by this engineer that several other prospective bidders had answered in a similar manner.

A letter* by Edgar A. Rossiter, Engineer, Department of Track Elevation, Chicago, Ill., states:

"In regard to the discussion over the new Genesee Ave. bridge at Saginaw, Mich., would state. * * * Bids should be asked on the substructure and superstructure separately, and then many reputable companies would be glad to submit detail plans of superstructure, as it would be right in line with their work and they can do so without excessive expense. There are many bridge firms who can obtain all the work they desire at the present time in superstructure alone, and do not care to figure on substructure, others will not figure where the substructure and general conditions existing are an unknown quantity; all these features reduce the number of bidders to a few, and then the lowest figure is based on a guess or inside knowledge of the facts. * * * "

Following up this line, the speaker took occasion to write to a contractor, who, he had reason to think, had probably bid on this Saginaw job. This is his reply, and it bears out the speaker's contention so fully that it is quoted in full:

"Replying to your favor asking our opinion as to the advisability of making separate contracts for different branches or trades involved in bridge work, will say that we think it is decidedly advantageous to the purchaser to let the contracts for the different trades separately; steel superstructure in one contract, masonry or concrete work in another.

"It is a fact that we frequently refuse to bid upon work for the reason that we would have to bid upon masonry as well as superstruc-

* *Engineering News*, February 13th, 1902.

ture. In cases where we do bid on the entire job complete, we always intend to bid safe and high enough on the substructure so we can turn around and sublet it to men who are engaged in that line of work. We think there is no question but what, as a rule, money can be saved the purchaser if contracts are let separately. Mr. Gifford.

"At the Saginaw, Mich., letting, which you refer to, will say that the substructure and superstructure were combined, and parties had to bid on the entire work. At the first letting we refused to bid upon the work, but did bid at the second or re-advertised letting."

These quotations show that this subject is being considered by others, as well as contractors, and if the present common practice of making blanket contracts, particularly on public work, which the speaker believes to be radically wrong for all parties concerned, could be reformed by those in charge of the letting of contract work, the speaker feels that it would be a benefit, not only to all contractors, but to the public.

T. Chalkley Hatton, M. Am. Soc. C. E.—The speaker looks upon this question from the standpoint of the municipality, or his client, and the result of his experience has indicated clearly that work can be done much better and more cheaply by letting it out under separate contracts, each representing the different branches of the trade, than under one general contract. It is true that where this method is followed, one contractor is constantly interfering with the work of another, and this interference often results in claims for extra time or remuneration, but the speaker has never experienced any trouble in settling such disputes with satisfaction to the several parties interested, and without any extra cost to his clients. Mr. Hatton.

In the construction of a system of sewers the speaker has been able to save from 5 to 10% of the cost of a general contract by having the municipality furnish the several materials, such as bricks, cement, pipe and castings, letting out the labor only to a contractor. In one of the cities for which the speaker has for a number of years been engaged as engineer, the street paving has of late years been performed by letting out the labor to a general contractor, the city purchasing the paving metal and foundation materials.

In the construction of water-works and electric-power plants, with which the speaker has been connected within the last five years, the contracts have been divided up, so as to represent the several branches of the trade, with the result of better and cheaper work; and it seems reasonable to look for such a result, first, as to cheapness, in that the bidder in estimating the work, certainly adds a certain percentage of the cost of his material as a legitimate part of his profits; second, as to better work, in that the engineer deals directly with the manufacturer, and can thus control the class of material far better than if he must deal through a contractor who looks upon the question only from the side of expense.

Mr. Hatton. It may be said that, the municipality or corporation being in the market but seldom for materials of certain classes, the manufacturer will not cut his prices as close as though he were dealing with a contractor who is his constant customer. In reply to this it might be said that when dealing with the contractor the manufacturer is not always sure of his money; whereas, when dealing with the municipality or corporation his money is assured upon completion of his contract, and thus he can afford to sell quite as cheaply; and besides, the engineer, if he understands his business, should be quite familiar with the prices of the several products which comprise any engineering work which he is called upon to design, and can thus protect his client.

The speaker cannot agree with Mr. Gifford regarding the contractor preferring to have the work divided into different contracts, as it has been his experience that more competition in bidding has resulted in letting the work out as one general contract, as the contractor has a better chance of making up his loss from one branch of the trade by his gains upon another. The speaker thinks that the best scheme, all around, if the engineer can have the confidence of his client and the manufacturer as well, is to go into the open market and purchase the various appliances comprising the work. He can do this, particularly in the electrical field, under his own name, and do it at the same price that the agent or middleman gets, and thus save his client the commission. This has been done by the speaker for the past two years in corporation work with a saving of from 3 to 8 per cent. In public work, however, this method would be a difficult one to follow, as it at once arouses the suspicion that the engineer is getting a commission; and besides, the usual laws provide that all public work shall be advertised and let to the lowest responsible bidder. It is true that dividing up the work into separate contracts adds very much to the burden of the engineer, but if the engineer is not desirous of assuming this extra burden he should not undertake the superintendency of such work.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E.—The speaker, judging by his experience of thirty-five years, would answer this question in the affirmative, except in cases where the engineer or architect can select an expert general contractor. The work certainly can be constructed at less cost if the various items are separated, and the work of generalization devolves upon the engineer of construction. Better work can also be obtained under such circumstances.

The unfortunate part of the proposition is that the owners, and employers of the architect and engineer, have the idea that that work for which they are willing to pay the general contractor a handsome remuneration should be done by the architect and engineer for practically nothing. The vexed question of unbalanced bids would

be eliminated, and the owner would be positively certain that he got what was specified. Mr. Darrach.

The present pernicious habit of architects in letting the work for the mechanical installation for public and private buildings to the general contractor (who knows as much of the subject as the unborn babe, and is about as responsible) cannot be too strongly condemned. The speaker expresses himself with some degree of feeling in this matter, as a piece of work which he designed, costing more than $125 000, was given to the tender mercies of a general contractor, and the speaker's supervision of the work withdrawn. The natural result of this was bad work, heart-burnings, law suits, and all the attendant evils.

S. Bent Russell, M. Am. Soc. C. E.—There are contracts of a certain class, with which engineers have to do, that require special treatment, and which may well be mentioned in discussing this question, although, in view of the previous discussions, it may seem almost like changing the subject. The speaker refers to machinery contracts, such as for steam-engines, for boilers and furnaces, for blowing engines, pumping engines, dynamos, mechanical filter plants, floating dredges, or other machinery. In such contracts it is frequently, if not usually, the case that the contractor is the manufacturer of a certain class of machines. Mr. Russell.

For example, the contractor is a builder of pumping engines. His whole attention is given to building pumping engines which will be a commercial success. In such a case there should be no middle men between the builder of the pumping engine and the owner or user of the machine. The builder and the user should be in as close connection as possible. This rule should be observed, not only for the good of the parties to the contract, but also for the good of the general public, in the long run, because the advancement of the art of building pumping engines will thus be favored.

The builder of the pumping engine must know how the engine operates in service, in order that he may see best where improvements can be made.

It has been only too often the case that builders of machines could not get accurate information as to the actual operation of their products. If the user of the machine is making a commercial success with it, he does not stop to make efficiency tests or scientific observations.

In most cases, the machine builder can more easily obtain knowledge from the machine user where the two are dealing with each other directly and without intermediaries.

May it not be said then: First, that, where the work is to be done by the builder of a special class of machinery, there should be as few middle men as possible; and, secondly, that one good reason for this

Mr. Russell. is that the machine builder may the better obtain accurate information as to the success of his work in all its details, and thus the greatest advancement of the art be obtained.

Mr. Easby. M. WARD EASBY, M. Am. Soc. C. E.—There seems to be one aspect of this matter which has not been touched on, and that is, the view most owners take. Prospective owners who desire their work executed by contract are always anxious to know the lump sum for which it can surely be done, and this the general contractor guarantees, after a manner; in fact, he furnishes a definite figure.

Even if the owners feel apprehensive about extras, they do not seem to fear them as much as an estimate made from scheduling all the bids of the sub-contractors.

The fact that the general contractor accepts the responsibility of making a complete structure or plant, and thus relieves the owners of expense in making the work of the various sub-contractors come together, is what makes his position so strong.

An instance has been given of a mechanical plant, of some 1 200 H.-P., in which the engineer had no extras, although he let out all the work by sub-contracts; but it is not stated that it is even the general experience of this engineer, that, under such circumstances, no extras occur; and it may be said that few are so far-seeing or perfect that they can prepare specifications which will draw the lines of definition between the work of the various crafts so that all will come together as a harmonious whole, without causing some differences, and consequent charges for extras. It may also be said that few general contractors expect to handle their contracts in such a manner that the cost to them will be no more than the sum of their various sub-contracts.

It may be that the engineer will feel that his specifications are sufficiently clear on any point in doubt, and he may take refuge behind some blanket clause stating that he is to be sole judge in cases of doubt, and thus compel a sub-contractor either to show open fight or do the work for the sake of harmony; but, frequently, the speaker has known a general contractor to pay an extra to a sub-contractor, because it was necessary for the completion of the general contract that some unforeseen work should be done; and, in such a case, if there were no general contractor, either the owners would pay the extra or the engineer would compel some unfortunate sub-contractor to do the work or have an open rupture, which the sub-contractor could not well afford.

There are many such engineers and architects, and though, by their foresight and ability, they seem to save the owners from paying for extras, yet they soon become known to the trades, and the bids of the sub-contractors are correspondingly high.

Almost every general contractor is his own sub-contractor for one craft, as, in the case of large buildings, he is a carpenter or a mason;

or, for a bridge, the general contractor will be a company which manufactures the superstructure only; but, would anyone expect such a general contractor to make no allowance for the contingencies that will arise, due to the various sub-contractors not making their work come together so that there will be no hitch? Mr. Easby.

It is a fact that, to make everything run smoothly on a large contract, a general contractor must be an executive of the highest order, and the engineer who expects to let all work by sub-contracts is adding to his purely technical duties those of the general contractor; and, as the general contractor inserts in his estimate items for the cost of his management and contingencies above referred to, the engineer should also be allowed to insert in his estimate, or schedule of sub-contracts, the same items. He should also be allowed extra compensation, to at least the extent of a portion, if not all, of the profit the general contractor would make.

Owners will look with doubt upon such items as charges for executive management and contingencies, and for the latter will rather think the engineer should be so proficient that there should be none; yet, in the case of the lump-sum bid of a general contractor, these items form a part. The owners, however, do not see the estimate in detail, and, not seeing such items, they are not called to their attention, and so they accept the lump-sum bid, trusting to their engineer to protect their interests.

In lump-sum bids the general contractor, under the stress of competition, strives to his utmost to get sub-contractors to shade as close as possible to the specifications, both in quality and quantity, and, to quote a victim of this system, "you get the least the worst man can do."

The system of letting work by general contract is founded largely on the fact that the designing and supervising engineer or architect is ignorant of how to execute his design; and, though he may know how it should be executed, he has little or no knowledge of how to go about it, and so the general contractor, who most frequently does not know how to design, but has plenty of assurance, steps in and relieves the engineer of the responsibility.

Engineers, in letting work by sub-contracts, should be allowed all the items of expense which the general contractors find necessary. In such cases, also, the owners are more at liberty to select their sub-contractors, and need not take the lowest one, in any particular class of work, if they and the engineer consider it to be unwise. The control of the work, in case of changes (especially where less is to be done than originally contemplated), is more to the advantage of the owners in the case of letting by sub-contracts.

Work let by sub-contracts is not so likely to be done for less than by general contract, but the class of work is likely to be infinitely

Mr. Easby. superior, and it is along this line that it seems best to urge the adoption of the system, which should have the support of all engineers.

Mr. Worthington. CHARLES WORTHINGTON, M. Am. Soc. C. E.—The speaker has never been able to see just why contracts for construction, made up of different and distinct classes of work, should be let as a whole, instead of being divided up into logical subdivisions and let as a number of separate contracts, excepting that it lightens the work of the engineer and permits him to shift the responsibility, for the work coming together properly, from himself, where it belongs, to the contractor.

There are a great many small contractors who are expert in their own line of work, but who know little about any other, whereas there are comparatively few who are experts in all classes of work. As Mr. Gifford has said, the bridge manufacturer cannot be expected to know all the details of masonry construction, and this holds equally true in many other cases. Now, if the contract is let as a whole, the former class is practically barred out, and competition is lessened to this extent; whereas, if bids are asked on the work properly subdivided and with each subdivision clearly defined, a maximum of competition will be assured, for each bidder will be thoroughly familiar with the particular work on which he is asked to give prices, and consequently will quote his best figure with a minimum percentage added as a factor of ignorance.

The engineer's place is that of intermediary between the purchaser and various contractors, and his duty to the former requires him to see that the most experienced workmen are employed on the several classes of construction, and that the total cost be kept at a minimum. The purchaser generally knows little about the cost and less about the work itself. For the best results, the engineer should be thoroughly familiar with both, and the contractors should be experts in their several branches. The engineer's plans and specifications should show clearly and completely every feature of construction, so that the several contractors may know exactly what they are bidding on, and so that possibilities of misunderstandings after contracts have been let be reduced to a minimum. Frequently, the engineer is allowed so little time in which to make his plans that this is not possible, in which case, of course, he must use his best judgment to make final results come out satisfactorily; and, under these conditions, the speaker advocates letting contracts on the basis of unit prices for the various classes of work covered by the contract.

Mr. Andrews. HORACE ANDREWS, M. Am. Soc. C. E.—Although the discussion thus far appears to be all in favor of the subdivision of contracts, there are reasons that may be urged in support of the opposite course.

The dependence between the different branches must often make it more desirable for one person to have the whole work under his control, even if some of its subdivisions are outside of his specialty and must be sublet to those better qualified for their execution.

This month, a large contract was let in Troy, N. Y., where the work was subdivided by the engineer into two contracts. The bidders who were the lowest for the work as a whole made the stipulation that they should have both branches of the work or none. They thought the relationship of the separate parts of the work was so close that the main contract could not be handled profitably or satisfactorily without control of the other. Nevertheless, the work has been let under two contracts, other persons being willing to assume the greater risks. Mr. Andrews.

Although it may often be to an employer's pecuniary interest to subdivide a contract into parts, each of which can be handled by a contractor specially qualified and properly equipped for its performance, still the subdivision of the work entails so great a possibility of conflict between the various contractors, so many opportunities arise for claims and concessions on account of remissness of some of the contractors, that it may be to the real interest of the employer to have dealings with one responsible contractor for the whole work, even at a slight pecuniary loss.

Lawmakers are disposed to ignore a contractor's technical qualifications, provided he is pecuniarily responsible, regarding it to be the duty of the engineer to enforce all requirements of the specifications, until the financial ruin of an incompetent contractor throws the completion of the work upon his unfortunate bondsmen. In such an event there is often less chance of loss to the employer from an attempt to compel responsible bondsmen of one general contractor to complete the work than from combating claims of numerous contractors executing separate branches of the work. Such contractors, on a subdivided piece of work, could reasonably claim to be damaged by delays occasioned by the suspension of work upon the execution of which the completion of their own depended.

Considerations of this nature may have influenced the framers of the law to which Mr. Gifford refers, whereby the Superintendent of Public Works of the State of New York is prevented from subdividing certain State contracts.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 932.

THE CONTROL OF NON-NAVIGABLE STREAMS BY THE NATIONAL GOVERNMENT.

An Informal Discussion at the Annual Convention, May 22d, 1902.

SUBJECT FOR DISCUSSION:

"In view of the numerous disasters caused by the contracting of channels, or the damming of small streams, should non-navigable streams be under the control of the National Government?"

By Messrs. RUDOLPH HERING, CHARLES G. DARRACH and FRANCIS COLLINGWOOD.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—This topic does not allude to the questions of diverting water from a water-shed, of silting up or of polluting streams; yet these are quite as important, and are pressing for solution. As they are also intimately connected with the damming and regulation of streams, it seems but natural that they should be considered in connection therewith, and the speaker accordingly takes the liberty to do so.

As civilization spreads over a country, changes take place in the physical character of the land surface, by the decrease of forest areas and the corresponding increase of cultivated areas, allowing the soil to be more readily washed into watercourses, and, in populated centers, by the addition of surfaces more or less impervious to water. Changes take place also in the character of the water as it flows in the streams, and in the size and shape of the beds themselves.

Mr. Hering.

The water when flowing is likely to change, both in quality and in the distribution of quantity. The more soils are exposed to washing by rain-water, the more soil particles will be brought into it, and carried in suspension by it, causing it to become more or less turbid. The sites of habitations and manufactories in city or country, and the application of manure to cultivate fields, cause every rain storm to bring into the streams organic matter which causes them to become more or less polluted, if not directly offensive, and injures the water, either for domestic consumption, for fish life or for manufacturing purposes.

The quantity of water flowing in a stream is changed by the growth of population in a way that flood flows tend to increase, and dry-weather flows tend to diminish. Flood heights increase by the reduction of forest areas and of water-retentive vegetation, which causes more rapid run-offs and greater velocity in the streams. Dry-weather flows decrease in consequence thereof, because there is less time for percolation into the ground, and therefore less storage of ground-water to feed the streams in time of drought. The use of water for irrigation purposes still further reduces the stream flow, both by direct diversion of the water and by evaporation on the surface of the land.

The beds of streams, so far as they are alluvial, change in accordance with the added amount of suspended matter carried by them, and also with greater floods and their consequent scour and re-deposit of material in lower stretches of the stream.

New causes therefore appear which tend to increase deposits when velocities become less, and to increase scour when these become greater. As both effects will not exactly compensate each other at the same points, the bed of an alluvial river is constantly undergoing changes which have increased since the advent of civilized man.

Stream beds are still further and arbitrarily changed by man when he endeavors to develop the usefulness of the watercourse, and builds dams across it, either for slack-water navigation or for power and irrigation purposes; and again when he builds or fills material into the stream, or in some way encroaches upon its natural flood section, contracting it without attention to other consequences.

All these alterations in the original conditions of a stream become the more annoying, sometimes even disastrous, by causing other results than those desired, the more a community becomes settled and developed. The objectionable results are due, perhaps, to the higher floods which now destroy more developed property, perhaps to the lower dry-weather flows, which deprive water users of some of their original ownership, and perhaps also to greater pollution of the water, tending toward unfitness for ordinary domestic, manufacturing or farming uses.

The multiplicity, as well as the seriousness, of effects upon large

Mr. Hering. public and private interests caused by the streams both in their natural and artificial conditions, due to a large growth of population, is clear; some of the effects relate to sanitation, some to property rights for domestic, farming and power purposes, and some to commerce.

The National Government now controls the streams of our country, as far as they are navigable. It can regulate, dam and improve them for commerce alone. It has no control of the streams for the purposes of sanitation, irrigation or farming, for domestic and power supplies, or for any purpose of damming or contracting those that are non-navigable; nor, perhaps most important of all, has it control over the regulation of streams to prevent disaster from floods. And yet, in a developed community, these effects, while of small consequence in its early life, become of great consequence in its later life.

Reports are becoming more numerous of the bad effects of floods caused by the improper construction of dams or the change in the physical conditions upon the water-shed surface, of disasters caused by a narrowing of the flood channel or by accumulated deposit therein, or of polluted streams which have become injurious to the health and life of man and cattle.

The subject has become so aggravated in a few States that official bodies have been appointed and instructed to grapple with it in one or another of its phases.

State commissions will answer perfectly when the causes and effects are within the State. There is a clear difficulty, however, in having a single State endeavor to regulate the natural and artificial conditions of one of its rivers, when this originates in another State and is received in an injured condition, perhaps polluted, perhaps with its dry-weather flow seriously diminished or its floods increased. The other State may exhibit very little interest in the matter, and its legislature may decline to effect either the desired remedy or any remedy that had so far been suggested.

It is palpably unjust to a State, as it is to an individual, to permit a change to be made by one party to the injury of another, in the enjoyment of the natural privileges granted us by the land where we live, without a careful, impartial examination of the facts and an impartial solution of the difficulty.

While the United States Supreme Court is a proper final authority in many such cases, no court can ever be constructive, in a physical sense, and therefore will not answer the purpose now before us.

We need for this purpose in our country some Board or Commission of Waterways, which should have supreme authority over matters relating to the physical conditions of a river from its source to its mouth, irrespective of State boundaries, consistent with and resembling the authority already placed in national hands for regulating nav-

igable streams for commerce; an authority which can make a thorough investigation into the physical conditions, not only collecting and applying information now in the hands of various public departments, but supplementing the same with local information of special value; to study the entire subject relating to floods and droughts, and pollution and diversion of the water; to establish official profiles and cross-sections for both dry-weather and flood conditions, to which all private interests should conform; and to establish limits for the pollution and diversion of water. Mr. Hering.

The engineering work of such a body would be considerable, yet the lines along which it should be undertaken would be clear, and the results like those of similar engineering enterprises. The organization of such a body, from a legal and political standpoint, is equally important, and is a factor, perhaps, more difficult of solution. It will not be touched upon here.

The programme of the topic under discussion asks whether non-navigable streams should be under the control of the National Government. In this opening the speaker desires to take a broader view, and would add: "or of Joint State Commissions, each one controlling the water-shed of at least one large stream?" The speaker does not in the least feel opposed to a control by the National Government; in many ways it would appear to be most satisfactory. There are also good and practical reasons for having, instead, a separate commission for each large water-shed, or group of smaller ones. A local commission would be better informed concerning local conditions, which are or may be highly important, than if most all of its members resided elsewhere. In either case, it would be desirable to place upon such a board a member of the United States Engineer Corps, and a member from each of the Departments of Agriculture, the Coast and Geodetic Survey, and, perhaps, also from a Bureau of Commerce.

It seems sufficient at this time to call attention to the importance of a proper regulation of our non-tidal streams, of the physical practicability of doing this, and of the necessity of having such regulation undertaken by a body whose authority must extend over the territory of at least one entire water-shed; our whole country being divided into as many water districts for this purpose as may seem best.

As precedents for such an undertaking we may consider the following:

The Mississippi River Commission is a body which has authority over the regulation of our largest river, extending through several States. Its authority, however, is less broad than it appears, from what was said above, might properly be given it.

Recently, a Flood Commission was established in the State of New York to report on flood preventions. The work of such a commission

Mr. Hering. must be hampered to some extent by its limitations, and, unless it takes a broad view, and has authority and means to make comprehensive investigations and carry out recommendations, its usefulness to the State cannot be of the extent which is here contemplated.

In England, due partly to the more uniform and smaller rainfall and stream flows than ours, and partly to the previous attention that has been given to the question of damage by floods, these matters do not now receive very serious attention. The large industrial development, on the other hand, has compelled the enactment of laws for the government of streams, prohibiting the introduction into them of solid matter not carried in suspension, such as earth, ashes, building rubbish, sludge and solid sewage, also all liquid sewage and polluting liquid from manufactories, provided that remedies are reasonably practicable and available under the circumstances of the case. The carrying out of these laws is placed in the hands of River Conservancy Boards and Joint River Committees, each of which has control of one or more water-sheds, and in some matters acts in conjunction with the Local Government Board, which has general charge of such matters.

In France, for many years there has been a special bureau in the Department of Public Works called the Hydrometric and Flood Announcement Service (*Services Hydrométriques d'Annonce des Crues*). This bureau has studied the general conditions of each river basin, the means to prevent inundations and to regulate and equalize the flow, and sends advices when floods are expected. The water-sheds for which special Boards were instituted under the direction of the Chief Engineer of Bridges and Roads (*Ingénieur en Chef des Ponts et Chaussées*) are: Seine, Canal de la Sambre, Escault et Yser, Rhône, Muerthe et Mosel, Aune, Tech et L'Agly, Garonne, Dordogne et Adour, and Loire. Among these, the Seine, Rhône, Garonne and Loire are the most important. Quite recently, a law has been passed authorizing the Department of Hygiene to inquire into the pollution of rivers, and provide remedies. It also authorizes the protection of all sources of water supply, and imposes fines for polluting the same.

In Germany, the regulation and correction of streams is in the hands of Provincial Governments in Prussia, and the State Government in Bavaria. The smaller states agree to the treatment of interstate rivers by the adjoining countries before work is done.

Each of the Prussian provinces has a Stream Building Commission (*Strombaudirection*) with a Chief Engineer (*Oberpraesident*) at its head, which controls design, construction and operation for each river flowing through several districts, and even into adjoining provinces, so as to obtain uniform results for the entire river. Such Commissions are located in Dantzic for the Vistula, in Breslau for the Oder, in Magdeburg for the Elbe, in Coblenz for the Rhine, in Munster for the Dortmund-Ems Canal, and in Potsdam for the Havel, Spree, Oder-Spree

Canal, etc. The mouths of the rivers are controlled by another branch of the government, for political reasons. The rivers pollution questions are under the control of a Minister of the Government, advised by the Imperial Board of Health. Mr. Hering.

For information concerning European countries the speaker is indebted to Ernest Pontzen, Cor. M. Am. Soc. C. E.; Theodor G. Hoech, M. Am. Soc. C. E., and R. A. Tatton, M. Inst. C. E.

In some respects, it will seem better to have separate authorities to regulate the questions of quantity and quality of the water, speaking broadly. On the other hand, there are many intimate connections which point to a preference for a single commission controlling both hydraulic and sanitary questions. In Europe these questions are now in the hands of different bodies, both, however, branches of the general government.

With many of our larger rivers, flowing from one State into another, it seems proper and timely to obtain, as early as possible, a harmonious and intelligent treatment of their profiles and sections, and also of the questions of diverting and polluting their water. Official profiles should be established for the entire river, which will regulate the velocities, both at high and low stages, so that they will not cause disastrous results. Normal cross-sections should be established for the ordinary and also for the freshet flows. No obstructions whatever should be allowed within them, although the territory rarely flooded, but lying within the flood sections, might be used for agricultural purposes, as is sometimes done in Europe, with the risk of occasional destruction of crops. As it may often be necessary to protect wide expanses of adjoining property, some flood sections would have to be diked; and, to prevent interruption of the crossing traffic, the entire flood section between dikes would have to be bridged at the proper height. In many cases it would be advisable to alter the alignment of the stream more or less, to straighten and shorten it, where now there is meandering. In adapting such changes to the special soil of the bed, it is often also necessary to construct special submerged dams, so as to maintain the original and natural regimen of the streams.

An important duty would also be the establishment of storage reservoirs where found expedient, for the purpose of reducing flood discharges and increasing low-water flows, both of which may benefit the riparian owners by preventing damage in one case and ensuring greater usefulness in the other.

The diversion of a part of a stream should be regulated in such a manner that the water, in accordance with its value, could be properly and justly distributed among the respective States within its watershed, and deprive no State of water to which it has a natural right.

Where works of industry, and particularly of ore-washing, load a stream with matter that is first carried in suspension and then depos-

Mr. Hering. ited lower down, thus raising its bed, proper means should be proposed to prevent injury to any riparian rights without unjustly interfering with important industries.

And lastly, such an authority as above suggested could establish regulations regarding the pollution of water from source to mouth, and thus protect it for the benefit of all users alike, whether in one State or another. It would devolve upon the same authority to determine the proper uses to which the waters of certain rivers could be put. In some cases it would be practicable, and even necessary, to reserve them for the domestic supplies of future populations; in others, manufacturing interests may abound to such an extent that certain streams should not be devoted to domestic use, but reserved for other uses, and protected only to the extent of not becoming objectionable to sight or smell.

Regarding the important matter of payment for the work herein considered, it may only be said that whether the National Government or Joint State Commissions undertake the same, it would manifestly be unfair to pay for it entirely from funds of the General Government, in districts where it is not itself a large landowner, because the benefit would accrue generally to the water-shed affected, and particularly to the riparian property owners. Payment, either by the State at large within which the improvement is made, or by the assessment of benefits against the counties or the private individuals affected, would seem more equitable.

All the subjects here suggested, and but briefly touched upon, are more or less effectively solved in the countries of Western Europe. They are now becoming more and more urgent of solution in our own country. It is to be hoped that we will not be long in arrears in crystallizing the method of handling this important subject in non-tidal streams as well as it has been handled in tidal rivers, in part by the United States Corps of Engineers, and that the results may be at least as beneficent as those accomplished across the Atlantic.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E. (by letter).—The control of non-navigable streams involves their entire course from source to mouth. There is but little doubt that the regimen of streams passing through two or more States can be controlled by the National Government, but the individual State alone has jurisdiction over non-tidal streams wholly within its own boundaries, unless it can be shown that lack of vigilance on the part of a State works danger and damage to the health and prosperity of the citizens of another State with whom the delinquent is indissolubly united.

Present knowledge makes it possible, with slight expense, to prevent absolutely sewage pollution of streams, and the subject under discussion should take into consideration such prevention. In the writer's opinion, the aid of the General Government can be invoked

to conserve the purity of any non-tidal stream, even if its entire course lies wholly inside the confines of an individual State, without violating State sovereignty, the only bulwark against imperialism. Mr. Darrach.

A State must be considered as an individual or a household in a community is considered. The individual and the household, although imperial, must be governed by the general laws of the community.

Pathogenic bacteria may be carried in the body and be voided from a perfectly healthy person, and be the source of spreading disease; so that a healthy individual may be the vehicle for carrying disease from the polluted stream of one State into that of another; and if a State refuses to preserve the purity of streams lying entirely within its own borders they may become sources of disease and danger in other States.

This subject is most important, as it deals with the life and health, as well as the comfort and wealth, of our entire country.

At present the War Department of the National Government has control of the national engineering. Does not the subject under discussion point to the advisability of a new department in our National Government dominated by civil engineers?

Francis Collingwood, M. Am. Soc. C. E. (by letter).—This subject is one with which the writer has had intimate acquaintance in connection with the streams in the "southern tier" of New York State; and, as emphasizing the remarks of Mr. Hering, he desires to cite the valley of the Chemung River, with which he has been acquainted for more than half a century. Mr. Collingwood.

A number of years ago, after a disastrous flood, which reached a higher point at Elmira than any that ever preceded it, he was called upon to devise means by which the city should be protected from future flood damage. A careful survey was made for about 3 miles of the length of the stream, with cross-sections of the stream and portions of the valley at frequent intervals. Having removed from the vicinity some twenty years before, the writer was amazed at the changes wrought in that time.

At a point about a mile below the center of the city the stream had widened from a normal width of about 600 ft. to some 1 400 ft. The banks, formerly steep and covered above with grass and trees, were cut and gashed, and there were small detached pools of water, and a growth of black alders reaching out 100 ft. in width. The stream bed, formerly clear and with no obstruction, was now a series of little gravel islands from a few square feet to several acres in area, and also covered more or less with alders. In place after place every particle of the black mould had been removed from the ground, leaving a sterile waste of clay, covered with coarse weeds. This destruction was not limited to one locality; the same conditions were

Mr. Collingwood. to be observed everywhere along the stream, and the writer was informed by a large land-owner at Wyalusing, on the Susquehanna, that the river flats, which formerly he considered his most valuable land, were now almost worthless for agricultural purposes.

The causes, in the case of the Chemung River, are not far to seek. One of the streams forming it is the Tioga River, which has a very steep descent. Formerly, the region was heavily wooded, but the timber has been entirely removed. The results are shown strikingly in one fact, viz., after a storm the floods now reach Elmira in little more than half the time they formerly occupied. The history is familiar to all intelligent observers, and need not be dwelt upon. The worst of it is that heavy floods are increasingly numerous, and the damage continuous. The converse of this, that is, the very low water in summer at present, as compared with the clear, bright, flowing river of former times, is also in evidence.

Now, as to the remedy: The design proposed was to restore the sections of the stream, by diking and dredging, in such way as to pass the largest known flood safely, and without damage to property.

The objections were, first, the cost, estimated at about $700 000; second, the fact that to this must be added a constant yearly assessment for maintenance. The improvement proposed that the material excavated should be utilized in building on each side of the river a pleasure drive. A third objection, in the writer's mind, was that the concentration of the full flood at the lower end of the improved channel, instead of its being spread over the valley as it is at present, might lead to damages to regions below, which could not be foretold.

The only result of the survey was the building of one or more dikes, which seem to have been seriously damaged in a recent flood.

A copy of the report was sent by the writer to the Governor of New York, and his attention was called to the enormous and irrevocable destruction of valuable land, and also to the fact, as outlined above, that no separate municipal or other body could with propriety undertake stream control. It could only be accomplished successfully by the State. Another remark of Mr. Hering is here to the point; the Tioga River heads in Pennsylvania, while the Chemung River is in New York.

In an article in the "Journal of Forestry and Irrigation" the effect of deforestation was shown by the record of actual observations made during the eight months succeeding April 1st, 1901, on Salt River, Ariz., below where it is joined by Tonto Creek, and on Tonto Creek half a mile above the junction. The basins are contiguous and similar, but the Tonto Basin is heavily grazed and almost bare of timber, grass or other vegetation; while the Salt River Basin lies in the Indian Reservation, where the sheep and cattle of the white men are not allowed, and it is heavily timbered and well carpeted with grass. The Creek

carried 0.00275 of 1% of sediment, while Salt River carried only 0.00146 of 1%, or only about half as much. Mr. Collingwood.

This brief statement is presented, for the reason that the facts lie wholly within a lifetime, and show how rapidly the destructive agencies act when once set in motion. There are indications that the same cycle of events is in progress in the extreme West, where some of the most heavily wooded regions are being rapidly deforested. The suggestions of Mr. Hering are wise, and to the point.

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IRRIGATION WORKS.

An Informal Discussion at the Annual Convention, May 22d, 1902.

SUBJECT FOR DISCUSSION:
"Should the National Government undertake the construction and operation of Irrigation Works?"

By Messrs. ELWOOD MEAD, T. M. RIPLEY, F. H. NEWELL, GEORGE H. MAXWELL, J. JAMES R. CROES, L. M. HAUPT and CHARLES G. DARRACH.

Mr. Mead. ELWOOD MEAD, M. Am. Soc. C. E.—There are two classes of irrigation works which the National Government may properly build and operate. The first are reservoirs located in the channels of streams the waters of which are used in irrigation; the second are diversion works of great magnitude and cost. Both promote the public welfare in such manner that public opinion should favor the extension of such aid.

Reservoirs are a necessity to the largest and best use of the water supply of the arid region. There is scarcely a stream which can be fully utilized in irrigation without storing a part of its flow. This is due to the fact that streams do not rise and fall with the demand for water; they are high when but little is used and low when water is most needed and most valuable.

While the mountain snows are melting floods run to waste, but when the snows are gone streams shrink to a mere shadow of their former volume, and this shrinkage occurs when the needs of irrigators are greatest. The studies, now being carried on by the Irrigation

Investigations of the Office of Experiment Stations, of the duty of water, when compared with the flow of streams, as shown by the gaugings of the United States Geological Survey, reveal the fact that a large part of the water supply runs to waste before it can be profitably used. Weber River, in Utah, has dropped from 4 588 cu. ft. per second in June to 455 cu. ft. per second in July. The high-water flow of the North Platte River in June has been thirty-four times as great as its low-water discharge in August. The discharge of Clear Creek, one of the irrigation streams of Northern Wyoming, in June, 1898, was 32 000 acre-feet; in July it was less than 12 000 acre-feet. Mr. Mead.

Before these facts were understood, many ditches were built to utilize the floods of spring and early summer. Much of the land under these ditches is now either idle or its cultivators often suffer severe losses. The construction of reservoirs is the only means of improving these unsatisfactory conditions. Such reservoirs will not only increase the farmed area, but add to the profits and security of the farmer, and will be worth far more than they will cost. Why, then, should their construction not be left to private capital? The answer is: To avoid legal and economic complications over water rights. Storage works located in stream channels must receive all the water which comes down from the mountains above. They intercept the water when the river is low, when it is all needed and all belongs to those having vested rights. Many of these reservoirs will be in places remote from the lands now irrigated. The farmers who need all the natural flow cannot see what is taking place, and, when they suffer from shortage, the reservoir which stands between their farm and the mountain snows, if it is a private property, will almost inevitably be held responsible. Distrust and anxiety thus roused will lead to controversy, if not litigation. All these evils will be less likely to arise if the works are public. Irrigators will be less likely to suspect interference with their rights when the works are under public control than when they are operated for private gain.

There are other reasons why these works should be public property. Where storage works are built and controlled as private enterprises, there is danger that the owners of such works will trade upon the necessities of farmers, and, when their fields are suffering from drouth, will charge such high rates for water that the prosperity of irrigated agriculture will be endangered. But if these works are built by the Government, the tendency will be to make water rates reasonable and stable. The doctrine of public ownership can be maintained over the stored supply as well as over the natural flow. Monopoly of water will be impossible, and its abuses averted.

The argument for public reservoirs, therefore, is not that private capital cannot afford to build them, because recent experience has shown that many of these works can be made highly profitable, but

Mr. Mead. that they should be regarded as a public utility, just as we regard bridges across streams on public roads. There is no question that toll bridges on many highways would pay, and there is equally no question that while they would be profitable to their owners they would be a nuisance to every one else. That, it is believed, would be the experience with many private reservoirs located on running streams.

The construction of costly diversion dams and main canals on large rivers, by the Government, is justified by the fact that the United States owns large tracts of land, now practically worthless, but which could be made habitable and productive by this outlay. If ownership carries with it responsibility, then it is the duty as well as to the interest of the Government to administer this property so as to make it provide the largest possible opportunities for those seeking homes on the public domain. To say that the Government shall take the attitude of an alien landlord who makes no improvements and pays no taxes, is not an enlightened conception of its duty, nor is it just to the arid States.

Neither individuals nor corporations will undertake these works unless there is a prospect of a profit commensurate with the risk. If this profit is realized it will put the cost of these lands beyond the means of poor men, who are the ones to be looked after in the disposal of the public domain. Only the well-to-do can buy land which costs from $15 to $25 an acre for a water supply.

The Government can better afford to do this work than can private enterprises, because the Government derives an indirect return which private enterprises cannot share. Every acre of land reclaimed adds to the taxable wealth and to the greatness and strength of the Nation. These benefits will be unending, and will increase with time. They are worth the outlay required.

It is proper that the Nation should build these works, because some of them will be located in one State while the water will be used in another State. It will be a much simpler arrangement to have this work done by the National Government than for the States to reach an agreement. Some of the States are too poor to utilize their resources, some are prevented by constitutional restrictions. The National Government can secure the needed funds from the sales of public lands, and build these works without any burden to taxpayers, East or West. The States have no such resource. They cannot even tax the public lands, which in some States comprise more than 80% of the total area.

There are also some vexed questions relating to the respective spheres of State and National authority over the control of streams used in irrigation. Each State now controls the division of water within its borders, and should continue to do so, but there is need that

this control shall be more effective, in order that controversies may be averted and the rights of actual users of streams made secure. In order to do this, the field of control of the Nation and of the State must be known. We are beginning to realize that the framing of water laws is among our most vital and perplexing problems. Many of the streams of the arid region finally flow into navigable rivers, and the question arises: Is the steamboat below or the orchard and garden above to have first claim on the mountain snows? There are interstate streams where on one side of a State boundary the doctrine of appropriation prevails, and on the other side the common-law doctrine of riparian rights. How are they to be reconciled? These questions will sooner or later press for a final settlement, either through legislation in Congress or by litigation in the Supreme Court of the United States. They involve the best use of resources of great value, and vitally affect the material development of one-third of the United States. Mr. Mead.

The construction and operation of irrigation works of the character above described, by the Nation, will tend to promote the enactment of more uniform and better laws by the States; it will help to educate public sentiment regarding the legal and economic problems which irrigation development is creating, and, in the end, make the arid region a more valuable and prosperous part of the Nation than if development is left wholly to private enterprise.

T. M. Ripley, Assoc. M. Am. Soc. C. E.—In speaking of this subject, at first thought, it seems that the fundamental idea is the wide-open policy of our Government in regard to emigration. Knowing this, and having lived for five years on the eastern slope of the Rocky Mountains, in Montana, there seems to be but one answer, viz., "Yes." Yet this answer must be qualified to include only that portion of the question which would deal with the building and operation of storage reservoirs. Such reservoirs would be interstate in their character, from the fact of their influence, not only on that portion of the stream above, but also on the government of the flow below them. Mr. Ripley.

The cost of such works, the fact of their influence passing state boundaries, the effect which, in many cases, they would have upon the flow of streams now under the control of the Federal Government, as well as the operation and maintenance of the same (they touch the very life of the land-owner in the arid region), make it not only expedient, but actually imperative, in some of these cases, that these works be those of the General Government.

On the other hand, it may and probably will be found more expedient that federal control should end at the river gate, and all ditches with their accompanying works be owned, operated, and maintained by those purchasing the water. Control of this water by sale, its proper apportionment to the land, and some restriction, so that the

Mr. Ripley. water cannot be bought for a speculative purpose, are absolutely necessary for the bringing of the maximum acreage under irrigation.

The quantity of water stored is always obtainable, the acreage possible to irrigate always known, and the former must serve as much of the latter as possible, to fulfill its duty.

What is to be done with the emigrants after receiving them within our borders, is a pressing question, the answer to which has heretofore been "Send them West." But the West is becoming filled up. Now, that may sound strange to a great many who know that they can ride for miles and miles along a river course, and even for days across country, seeing only a few log cabins; nevertheless, it is true. The country is becoming "filled up," for the simple reason that it has all the population it can at present support economically, from the very fact that there is not enough water, or that the water in the locality cannot be used.

Go to the headquarters of many of the mountain streams; follow them for miles, and what do you find? A few ranches located on the lowest of bottom lands, each ranch with a few irrigated acres; the remainder is "range," and, if the stream is small, few years go by without a "water fight." The man below, suffering from want of water, blames his neighbor further up stream, and this in the face of the fact that his fields are scarcely dry from the spring freshet of a few weeks before.

The Government is an owner and dealer in land, and, as was said before, it encourages the emigration of people to those lands. The current reports show that the number of emigrants will be greater this year than in any previous year, and to give a man an unirrigable quarter-section on which to build a home is like offering a stone for bread.

An article* by Mr. Elwood Mead begins: "Although one-third of the United States is arid, the importance of its reclamation and settlement to the Nation has thus far been disregarded."

Mr. Mead's statement is too sweeping. The Government has made surveys for dam sites and maps of the same, yet, so far as known, the matter has been carried no farther.

Moreover, legislation is now pending, and let us hope that something of real value to this large region may be accomplished. There are thousands of acres thirsting for the water which will place them among the most fertile in the world; square miles of virgin soil, the value of which will increase from 500 to 1 000%, when properly irrigated.

That the Government will build and own the great works which are necessary for the reclamation of this land, is, let us believe, the hope of everyone who is interested in our great arid region. Let the water, now making torrents and flooded rivers, be stored and sold at a

* *The Outlook*, April 12th, 1902.

just rate; then the land, now producing only sage brush, cacti and bunch grass, will be changed from cattle ranges into farms. Mr. Ripley.

Again, many good ranges and winter shelters have to be abandoned, and the cattle driven many miles to lower ground, or else may perish on the range for lack of food, when, with a proper storage and application of the run-off, the necessary amount of feed could be produced on the ground now abandoned. Should this work be once fairly started, it will so demonstrate its own value that nothing will stop it. It means a new West; a doubling of our resources.

F. H. Newell, M. Am. Soc. C. E.—The direct answer to this question lies now with statesmen, rather than with engineers, as such, since it is a matter which has been discussed from many standpoints for half a generation, and has entered into national politics to the extent at least of being embodied in the platforms of all the great political parties, and has formed a considerable portion of the recent Message of the President to Congress. Since the Chief Executive of of the Nation has answered the question in the affirmative it would not be in accord with official courtesy to take the opposite stand, even though it might accord with personal ideas, which in the present case it does not. Mr. Newell.

Under the circumstances, it might be considered preferable at the present time to discuss, not the question whether the Government should undertake the construction and operation of irrigation works, but rather, what are the limitations, or the extent to which the National Government should go into the matter. The developments within the arid region, through private enterprise, have proceeded to a point where the construction of large works, as a rule, is found to be impracticable from the financial standpoint. Many small works have been built and have been found to be successful, both in an engineering sense and in affording a satisfactory revenue; but the opportunities for building such small works have nearly all been utilized, and further development rests largely upon the question as to what shall be done toward storing floods or diverting large rivers.

The situation, as regards the vacant public lands and the opportunities for the reclamation of arid areas, has for many years been one of the matters for examination by the United States Geological Survey. It may not at first glance be apparent why this organization is concerned with the matter. The word "geology" to some engineers is synonymous with rock structure, or hypotheses as to the origin of veins and mineral deposits. The word, however, has a far wider meaning, and by geologists, at least, is made to include a thorough knowledge of the earth's surface, its history, present condition, and even the study of the natural resources upon and under the surface.

Mr. Newell. The Geological Survey is something more than the examination of the structure of rocks. In fact, its organic law demands that the Director shall have charge of the classification of the public lands, and examination of the geological structure, mineral resources, and products of the National domain. The necessities of land classification and mapping the resources have given rise to extensive surveys and engineering investigations, so that in classifying the officers or members of the Geological Survey it may fairly be said that fully one-half of this corps consists of engineers, as distinguished from geologists. The Director himself, although not claiming to be an engineer, has a well-earned reputation, not only as a scientific man, but, among an entirely different group of men, is known as one of the best designers and constructors of large buildings.

The explorations made by the Geological Survey in its early days attracted the attention of the public and Congress. In particular, the work of its former chief, John W. Powell (Director from 1880 to 1894), called attention to the importance of irrigation. In 1888 he was authorized by Congress to ascertain the extent to which the arid lands could be reclaimed by irrigation, to survey reservoirs and ascertain the cost of construction.

Elaborate plans for reclamation were made at this time, but the irrigation survey thus created had a short life, terminating in 1891, its work being continued on a small scale by what is known as the Division of Hydrography of the Geological Survey. Congress, however, apparently never gave up wholly the idea of Government work, since, in the various acts passed from time to time, the interest of the Government has been guarded, and in all land titles right-of-way for works constructed under the authority of the General Government has been reserved, and reservoir sites have been segregated on public lands.

The amount of water available for storage and for diversion has been the subject of study year after year by the Division of Hydrography, and data bearing upon this question of ultimate reclamation have been brought together. At the same time, a study of the public lands has been made, maps have been prepared showing the location of these, and illustrating the vast extent of land in each state and territory still in the hands of the General Government.

As a result of the information brought to the attention of the people, there has grown up a popular movement for the reclamation of the arid region. This has found expression in the organization of a body known as the National Irrigation Association, composed of influential business men, not simply of the West, but of the eastern or manufacturing portions of the country. These men appreciate that the development of the western half of the country rests wholly

upon irrigation, and that the West when completely developed will be the best market for the products of the factories of the East and the cotton fields of the South. The men who have investments in railroads also appreciate that their prosperity is closely linked with that of the growing West, since the earnings of the railroads and of the men whose goods they haul are dependent largely upon the amount of material carried backward and forward between the East and the West. Mr. Newell.

Notable opposition to this movement has come from only one source, and that is the eastern grange. There has been a fear that by the building up of homes in the West the agricultural lands of the East will be depreciated in value. A careful examination, however, shows that there is no competition of farm products East and West, but, on the contrary, the crops raised under irrigation are of different character and find another market than that of the humid East or the well-watered Mississippi Valley. The actual condition is just the reverse of that feared by some of the eastern farmers. For every acre brought under cultivation in the West there will be an acre cultivated in the East to supply food to the laborers in the manufactories or on the railroads furnishing the western farmer with the implements and articles for his home and farm.

The agitation which has continued has resulted in the bringing forward of many bills authorizing Government construction, one of which has already passed the Senate, has been informally, but favorably, considered by the President, and is now before the House of Representatives with, it is claimed, a majority pledged to its support. The question, therefore, as to whether the Government shall undertake the construction and operation of irrigation works will probably be answered in the affirmative within a few days by the House of Representatives.

In the meantime the executive officers of the Government have not been idle in collecting information for use in connection with proposed works. Extensive surveys have been carried on and plans have been made for a number of structures, so that, whenever money is available, work can be begun toward carrying out the wishes of Congress. The various projects now under consideration, and the detailed plans for some of these, can be seen at the office of the Geological Survey, and it is hoped that engineers interested will visit the office and inspect the results of some of the work.

In addition, detailed facts concerning the present condition of irrigation development have been collected by the Census Office, another bureau connected with the Department of the Interior, but entirely distinct from the Geological Survey, excepting that experts from the latter have been detailed to the Census Office for specific purposes. The Eleventh Census, 1890, was the first to take cognizance of irrigation.

Mr. Newell. The Twelfth Census, 1900, has continued the investigation, and there are now in the hands of the Government complete schedules showing the farm areas and values, and the detailed crops for every farmer in the United States. In addition, schedules are on file for most of the irrigators of the West, and detailed statements concerning ditches, canals and investments made in irrigation works. Without going into the subject, it is sufficient to state that the total area irrigated has increased from about 3 500 000 acres in 1889 to more than 7 000 000 acres in 1899, or 103 per cent. The number of irrigators in the same time has increased from about 52 000 to 102 000, or 95 per cent. The total cost of construction of the irrigation systems now in operation has been estimated at $64 000 000.

There is a tendency to greatly exaggerate the extent and value of the present irrigation developments. The statements given out from time to time by various parties would lead to the conclusion that the arid region is far more largely reclaimed than is shown by these figures. The census examination, however, has been made and checked in a very thorough manner, the enumerators' returns having been gone over with great care and compared with information obtained through correspondence and field examination, so that they are supported by a wealth of collateral evidence. To illustrate this point: It is popularly stated that in Nebraska more than 1 000 000 acres have been brought under irrigation within the last ten years. The detailed enumeration by the census failed to find anything like this acreage, and the field examination and correspondence revealed the fact that thousands of acres of the lands claimed to be irrigated have never been actually watered. In fact, less than 150 000 acres have been irrigated successfully, or not much more than one-tenth of the amount claimed. In the same way the mileage and cost of existing ditches fades away under a careful scrutiny, and many of the conclusions drawn, as regards conditions in the irrigated region, are found to be fallacious under the careful scrutiny of impartial statisticians. This point is emphasized because it is sometimes asserted that irrigation development in the West has already been completed by private enterprise, and that the Government should have no further interest in the matter.

On the contrary, the Government is still the great land-owner, having control of from one-half to nine-tenths of the area of the arid states, and, through the recently adopted policy of forest reservation, it is taking active steps to preserve and perpetuate the water supply, and, through the legislation now in hand, it is probable that it will continue in the line of reservoir construction and diversion of large rivers.

Mr. Maxwell. GEORGE H. MAXWELL, Esq.*—This discussion brings up two points to which attention should be directed.

* Executive Chairman, The National Irrigation Association.

The question under consideration is: Mr. Maxwell.

"Should the National Government undertake the construction and operation of irrigation works?"

When that question has been decided in the affirmative, the next question, and a very important one, is how to get the National Government to do it.

When the matter is presented to Congress, the question is immediately asked: What return will the Government get for the expenditure of its money? In answering this question it is essential to bear in mind that there are two specific propositions to be considered, and that they must always be kept separate and distinct, and never confused.

One question is: Should the National Government build the storage reservoirs to conserve the flood waters that now go to waste, in order primarily to regulate and equalize the flow of the rivers?

The other question is: Should the National Government build the reservoirs and main-line canals which are necessary to bring water within reach of settlers, in order to make possible the reclamation and settlement of the arid public lands?

The National Irrigation Association answers both these questions in the affirmative, and states them separately, among the objects of the Association, in its Constitution, as follows:

"Section 2.–The preservation and development of our national resources by the construction of storage reservoirs by the Federal Government for flood protection, and to save for use in aid of navigation and irrigation the flood waters which now run to waste and cause overflow and destruction."

"Section 3.–The construction by the Federal Government of storage reservoirs and irrigation works wherever necessary to furnish water for the reclamation and settlement of the arid public lands."

President Roosevelt, in his message to Congress, recommends the adoption of a broad and comprehensive national forestry and irrigation policy by the National Government, and the distinction between the two propositions in question is clearly drawn in his message.

Concluding that part of his message which relates to forestry, he says:

"The forests are natural reservoirs. By restraining the streams in flood and replenishing them in drought they make possible the use of waters otherwise wasted. They prevent the soil from washing, and so protect the storage reservoirs from filling up with silt. Forest conservation is, therefore, an essential condition of water conservation."

Passing from the subject of forestry to the question of storage reservoirs for water conservation and river control, he says:

"The forests alone cannot, however, fully regulate and conserve the waters of the arid region. Great storage works are necessary to equalize the flow of streams and to save the flood waters. Their construction has been conclusively shown to be an undertaking too vast

Mr. Maxwell. for private effort. Nor can it be best accomplished by the individual states acting alone. Far-reaching interstate problems are involved; and the resources of single states would often be inadequate. It is properly a national function, at least in some of its features. It is as right for the National Government to make the streams and rivers of the arid region useful by engineering works for water storage as to make useful the rivers and harbors of the humid region by engineering works of another kind. The storing of the floods in reservoirs at the headwaters of our rivers is but an enlargement of our present policy of river control, under which levees are built on the lower reaches of the same streams.

"The Government should construct and maintain these reservoirs as it does other public works. Where their purpose is to regulate the flow of streams, the water should be turned freely into the channels in the dry season to take the same course under the same laws as the natural flow."

Then he takes up the question of the reclamation of the arid public lands, and considers it as a separate and distinct problem, as it always should be considered, and, in reference to that, he says:

"The reclamation of the unsettled arid public lands presents a different problem. Here it is not enough to regulate the flow of streams. The object of the Government is to dispose of the land to settlers who will build homes upon it. To accomplish this object water must be brought within their reach.

"The pioneer settlers on the arid public domain chose their homes along streams from which they could themselves divert the water to reclaim their holdings. Such opportunities are practically gone. There remain, however, vast areas of public land which can be made available for homestead settlement, but only by reservoirs and main-line canals impracticable for private enterprise. Their irrigation works should be built by the National Government. The lands reclaimed by them should be reserved by the Government for actual settlers, and the cost of construction should, so far as possible, be repaid by the land reclaimed. The distribution of the water, the division of the streams among irrigators, should be left to the settlers themselves, in conformity with state laws and without interference with those laws or with vested rights. The policy of the National Government should be to aid irrigation in the several states and territories in such manner as will enable the people in the local communities to help themselves, and as will stimulate needed reforms in the state laws and regulations governing irrigation.

"The reclamation and settlement of the arid lands will enrich every portion of our country, just as the settlement of the Ohio and Mississippi Valleys brought prosperity to the Atlantic States. The increased demand for manufactured articles will stimulate industrial production, while wider home markets and the trade of Asia will consume the larger food supplies and effectually prevent western competition with eastern agriculture. Indeed, the products of irrigation will be consumed chiefly in upbuilding local centers of mining and other industries, which would otherwise not come into existence at all. Our people as a whole will profit, for successful homemaking is but another name for the upbuilding of the nation."

The broad distinction between the two propositions, storage reservoirs for river control on the one hand, and reservoirs and canals for

the reclamation of the arid public lands on the other hand, is in the nature of a return which the Government will get for its investment, and the way in which that return will come back to the Government. Mr. Maxwell.

Where reservoirs are built for river control, the return to the Government will be indirect from the increased prosperity and population of the country, and the resulting increase in our commerce and in the general welfare of the people, which will correspondingly enlarge the general revenues of the Government. It is from this indirect source that the Government gets the return which warrants the construction of any other river or harbor improvements, such as the reservoirs which have already been built on the head waters of the Mississippi River, the levees on the Mississippi River, the Eads Jetties at the mouth of that river, the Sault Ste. Marie Canal and Locks, and many other instances of river improvements which might be given.

Where reservoirs or large main-line canals are built to bring water within reach of settlers on the public lands, the Government would get its return directly from the lands reclaimed. The public lands to be irrigated would be reserved for actual settlers only under the Homestead Act, and each settler would pay to the Government his relative proportion of the cost of the irrigation system. This would be a charge on the land, just as in the case of an assessment for a local public improvement.

If the proposed National Irrigation Act now pending in Congress should be enacted into a law, it would carry into effect the policy advocated by the National Irrigation Association, and recommended by the President in his Message, for the reclamation and settlement of the arid public lands, through the construction, by the National Government, of the great reservoirs and main-line canals necessary to enable settlers to irrigate and reclaim those lands.

Under this proposed act, this would be accomplished without imposing any burden of taxation at all upon the people at large. The act proposes to set apart the proceeds from the sales of public lands in the arid and semi-arid States as a revolving construction fund in the Treasury of the United States to be expended by the Secretary of the Interior in building irrigation works for the reclamation of other public lands. When these other public lands have been reclaimed, and the cost of their construction has been returned by the settlers who have secured the lands, the money goes again into the reclamation fund to be reinvested in the construction of new irrigation works.

The amount which would be available, under the act, on June 30th, 1902, would be about \$6 000 000. To this would be added each year the estimated receipts from public lands, amounting to about \$2 500 000 a year. This would provide a fund of \$25 000 000 for construction in the first ten-year period. The average annual returns from public land sales would probably increase as the country settles up, because

Mr. Maxwell. of the greater demand for mineral land, oil land, timber land and grazing land. But, assuming that the annual returns continued at \$2 500 000 a year, there would be another \$25 000 000 from this source for investment in the second ten years, and the \$25 000 000 invested in the first ten years would come back for reinvestment in the second ten years, making a total of \$50 000 000 available for investment in new irrigation works in the second ten years. In the third ten-year period, another \$25 000 000 would be available from land sales, and the whole \$50 000 000 invested in the second ten years would come back for reinvestment during the third ten years, making a total of \$75 000 000 available for investment in new irrigation works in the third ten-year period. It is therefore a conservative estimate that \$150 000 000 would be available for investment in new irrigation works under this proposed act within thirty years from this time, and every dollar invested finally comes back to the Government.

Every safeguard which could be suggested has been embodied in this bill. It is essentially a bill for the benefit of the home-maker as against the land speculator. Lands to be reclaimed are to be withdrawn by the Secretary of the Interior, and cannot be entered except under the Homestead Act. The Commutation Clause of the Homestead Act is made inapplicable to land located under this act. Five years' actual residence is necessary before title can be secured from the Government. Non-resident owners, of lands under the system which have passed into private ownership, cannot secure a water right under any circumstances, and resident owners, or those living near the land and farming it, as in the village farm communities of Utah, can only secure a water right for 160 acres of land. Beneficial use is made the basis, the measure, and the limit of all rights to water under the act, and water rights are made appurtenant to the land.

As an illustration of what can be accomplished under this act, we may form some conception in imagination by looking at a single State. The State of Montana is as large as Illinois, Indiana and Ohio taken together. Those three States have a population of more than 10 000 000. Montana has a population of only about 250 000. If all the irrigable public lands in the State of Montana which could be irrigated from the water which now runs to waste were reclaimed by the construction of irrigation works, under this National Irrigation Act, Montana could be made to sustain as large a population as Illinois, Indiana and Ohio, and to produce equal to those States in their vast agricultural products. The Asiatic market would take every pound of surplus farm products from Montana, and no competition with eastern agriculture would be created. The demand for every manufactured product of our eastern factories would be enormously stimulated, and every section of the country would be benefited by the wide dissemination of the new wealth created in Montana. The Government would get back every

dollar that it invested in irrigation works, from the settlers who located the land. Mr. Maxwell.

It would be difficult for the human mind to devise anything which this government of ours could do which would promote the general welfare of the people as much as such a transformation as this of one of our western States. And when it is considered that Montana is but a single State, and that the whole West is capable of this same transformation, some idea may be formed of the vast potentialities of the national policy which will be inaugurated with the passage of this National Irrigation Act.

In the second question under consideration—the building of reservoirs for river control—the subject of the return to the Government must be approached from a different point of view.

As the President has said in his Message, the building of such storage works in the arid region would be merely an enlargement of our present policy of river control.

The River and Harbor Act of June 3d, 1896, contained the following item of appropriation:

"For the examination of sites, and report upon the practicability and desirability of constructing reservoirs and other hydraulic works necessary for the storage and utilization of water, to prevent floods and overflows, erosion of river banks and breaks of levees, and to re-enforce the flow of streams during drought and low-water seasons, at least one site each in the States of Wyoming and Colorado."

Under this appropriation, Captain Hiram M. Chittenden, M. Am. Soc. C. E., was detailed to make the investigation, and his report is known as the Chittenden Report.*

The report is a most exhaustive examination and discussion of all the conditions out of which the necessity for the construction of federal reservoirs grows, besides the detailed plans and estimates of the particular reservoir sites surveyed. One of these proposed reservoirs, near Laramie, Wyoming, has the enormous capacity of over 900 000 acre-feet of water.

After fully setting forth the reasons why it is impossible that these great reservoirs should be constructed by private capital or individual enterprise, and showing clearly the reasons why they should be built by the nation rather than by the State, the report closes with the following:

"The foregoing examination has led up to the following conclusions:

"First.—A comprehensive reservoir system in the arid regions of the United States is absolutely essential to the future welfare of this portion of the national domain.

"Second.—It is not possible to secure the best development of such a system except through the agency of the General Government."

* H. R. Doc. 141, 55th Congress, Second Session.

Mr. Maxwell. One of the strongest arguments in favor of the construction of such storage reservoirs by the Federal Government is clearly stated by Captain Chittenden in this report as follows:

"There seems to be a well-nigh universal consensus of opinion that the preservation of the forests of the arid regions is distinctly a Government duty. Considerable appropriations have been made for the surveys of proposed reservations, and ways and means for their preservation are being considered. Now, one of the great arguments always advanced in favor of forest preservation is the influence which forests are supposed to have in conserving the flow of the streams. Inasmuch as the commercial value of these forests is practically insignificant, except for furnishing fuel and rough timber, the water question is really the more important one. If it is properly a Government function to preserve the forests in order to conserve the flow of the streams, surely it can not be less a Government function to execute works which will conserve that flow even more positively and directly. Granting all that can be said of forests in this connection, they certainly can never prevent the June rise, and it is precisely this waste flow which reservoirs will help to save. The forests ought unquestionably to be preserved, and the Government is the proper agency to do it, but the principal arguments therefor apply with accentuated force to the construction of reservoirs."

No one familiar with the conditions in the West would question for a moment the proper and constitutional connection between such works of river improvement as we are now building and reservoirs to hold back the waters which now run to waste in floods. This can be illustrated by what the Government is doing on the Sacramento River, in California. That river is a navigable river. In 1849 and 1850 ships which had gone around the Horn sailed up the Sacramento River as far as Marysville. The channel of the river has been filled up with silt and débris from the hydraulic mines until flat-bottomed steamers are the only craft which can navigate it, and they have much difficulty during the low water of the summer season. There are two seasons of flood on the river, one during the winter months, from the rains in the mountains, the other during the spring, from the melting snows of the Sierra Nevada.

The National Government has appropriated and is now expending several hundred thousands of dollars in the attempt to control the débris problem and preserve the navigability of the Sacramento River. The State of California has appropriated a like amount, and the money is being expended under the joint supervision and control of the officers of the State and National Governments, who are working together in the most perfect accord and harmony.

The spring flood of the Sacramento River is usually the most serious. It comes very near to the tops of the high levees that protect the Cities of Marysville and Sacramento, and on the western side of the Sacramento Valley it overflows the banks of the river and broadens out through a wide shallow channel called the "Trough," and every year overflows and practically destroys the productiveness of many

thousands of acres of the most fertile land of the Sacramento Valley. Mr. Maxwell. Anyone who has traveled on the railroad from Sacramento to Woodland during one of the spring floods of the Sacramento River will remember the inland lake which is caused in that section of the country by this overflow.

There is no other way to control this overflow, and control the floods which will otherwise continue to bring down great volumes of silt and débris and fill up the channel of the river, or to preserve permanently the navigability of the Sacramento River, except for the National Government to take hold of the problem in a broad and comprehensive way and build great storage reservoirs in mountain valleys and in the basins among the foot-hills of the Upper Sacramento Valley, coupled with great main-line canals, skirting the foot-hills on each side of the Sacramento Valley, through which these flood waters can be drawn off and controlled.

A great overflow channel with well-defined banks could be constructed through what is now called the "Trough," so as to confine the water within fixed boundaries in that section. This artificial channel through the "Trough" and the main-line canal on the western side of the valley would debouch into the salt marshes of Suisun Bay, and all the silt and débris and sediment, instead of flowing down the river channel and destroying its navigability, could be deposited over wide areas of these salt marshes, which would gradually by this means be reclaimed and made productive for agricultural or grazing purposes.

The cost of this work, of course, would be great, but the speaker believes that the benefits from it, to the National Government, in proportion to the amount expended, would be as great or greater than those from any other public work of this character ever built by the Government. The cost of such works must always be measured by the results. The Sacramento River Valley is one of the richest and most fertile regions of the earth. Climate and soil have there combined, over the greater part of the valley, to create every condition necessary for the support of a dense and prosperous population, if the problem of the control of the Sacramento River and its floods can be solved.

It is quite true that the lands which would be benefited by these works would not be public lands, any more than were the lands which were benefited and protected from floods by the Government levees along the Mississippi River. The conditions on the two rivers, so far as the relation of the National Government to the problem is concerned, are very similar. The benefits to the National Government would be indirect, but they would be more than adequate to justify this great public improvement.

The State is powerless to act alone in the matter, because the

Mr. Maxwell. Sacramento River, being a navigable stream, is under the control of the National Government. But there is no doubt, if this work were to be undertaken by the National Government, that the State would co-operate with it and bear a share of the cost of the works, just as it is now bearing one-half the cost of what is being done to control the débris and preserve the river channel by make-shift methods.

The idea of a large main-line canal from the Sacramento River, leaving the river at the head of the valley and skirting the foot-hills on the east, is not a new one. A commission of able engineers was appointed by General Grant, in 1874, to investigate the irrigation resources of California, and the map published with their report shows such a proposed main-line canal, extending clear down to the lower end of the San Joaquin Valley. Such a canal in the Sacramento Valley would serve a double purpose. It would aid in controlling the floods of the river, and in the protection of its channel, and water from the canal could also be used for the irrigation of lands in the valley. Instead of this being an objection, from the point of view of creating reservoir capacity, it is an added reason for the use of the water. An irrigated region of country, when it becomes saturated with water, is in fact nothing more or less than an immense reservoir. The water taken out in the canal seeps into the ground, saturates it like a great sponge, and feeds it gradually back into the stream or river below, in the summer season, at the time when the water is most needed for navigation.

The amount of water which land will hold in suspension may be illustrated by an incident in the Salt River Valley in Arizona. Experiments, made at the agricultural experiment station there, show that in sandy soil a peach orchard absorbed during the winter months 10 acre-feet of water, which soaked down into the ground and so saturated it that the orchard needed no summer irrigation. The Tonto Basin Reservoir on Salt River is one of the largest and finest reservoir sites probably in the world. It will hold approximately 800 000 acre-feet of water; and yet that whole 800 000 acre-feet of water could be soaked into less than 100 000 acres of the lands of the Salt River Valley.

The remarkable results of the return seepage from lands saturated with water used for irrigation are shown in many places in the West. After the country becomes thoroughly irrigated there is a return seepage into the stream, and this establishes a permanent summer flow which it would be almost impossible to secure in any other way.

If, therefore, the possibility that the waters stored by the National Government on the Sacramento River might be used for irrigation were to be urged as an argument against the building of such storage works, such an argument would rest upon ignorance of the conditions of the country and the results of irrigating lands. Instead of de-

tracting from the value of a system of reservoirs built primarily for Mr. Maxwell. the improvement of navigation, and to regulate the flow of the river and preserve its channel, the use of the water for irrigation would improve and increase the effectiveness of a comprehensive storage system for these purposes on that river.

If it is conceded, as all do now concede, that the Government has the power, and that it is the proper function of the National Government to build débris dams, to dredge the channels, to excavate cut-offs, to remove snags, to straighten channels, and to do the many things which it is now doing in its efforts to preserve the navigability of the Sacramento River, surely there can be no question as to its power, or as to the advisability of its going to the root of the evil and removing its cause, by building a comprehensive storage system, which would in fact and in reality accomplish the desired end and which cannot be accomplished by present methods. And, if that be so, the speaker cannot see how it can be seriously argued that it is not a proper enlargement of the policy of the Federal Government, in the regulation of rivers, to build storage reservoirs. That fact being conceded, then the whole question becomes one merely of advisability and of the relation between cost and benefit.

Now, so far as concerns the returns to the Government, for moneys expended in that way, the proposition must be looked at from the broadest point of view.

Taking the whole western half of the United States as an illustration: Ten years ago the census showed a population of 4 404 000 in the western half of the United States, west of the 98th meridian, and in the eastern half, 58 218 000. The census of 1900 shows a population of only 5 874 332 in the western half of the country, and, in the eastern half, 70 120 243.

President Roosevelt, in his Message to Congress, stated, and no one can gainsay the fact, that:

"The western half of the United States would sustain a population greater than that of our whole country to-day if the waters that now run to waste were saved and used for irrigation. The forest and water problems are perhaps the most vital internal questions of the United States."

If the broad and statesmanlike recommendations on this whole subject, by President Roosevelt in his Message to Congress, are wisely carried out, there will be as large a population in the western half of the United States in less than fifty years from to-day as there is to-day in the whole country. The population is increasing by leaps and bounds. At the average rate of increase of the past, the population will be doubled, and there will be more than 150 000 000 people in this country within thirty years. And no less a person than Ex-Secretary Gage stated, a few months ago, in a public address at Denver, that there would be a population of 190 000 000 people within fifty years.

Mr. Maxwell. Now, what revenues could the Government get from 75 000 000 people in the western half of the United States? The people cannot be put there unless the recommendations of the President's Message are carried out. If those recommendations are carried out, they can and will be put there. Now we are collecting, and have collected, in this country a revenue amounting to very nearly $10 *per capita* on a population of 75 000 000 people. Suppose 75 000 000 people are put in the western half of the United States, by carrying out a wise and comprehensive national policy of forestry and irrigation.

The National Government could collect from them in one year more money than it would ever be called upon to expend for the complete carrying out of the whole vast undertaking of the reclamation of the arid region.

There is a point which generally seems to be lost sight of, but which should always be borne in mind, in considering the question of a return to the Government from water-storage works built as internal improvements for river control, where the cost cannot be returned from the lands reclaimed. Wherever a new population is created a new basis for national revenues is created, not only for one year but for all time to come. And it is perfectly safe to say that wherever the Government builds such works as are proposed in the Sacramento Valley it would create increased commerce, population and wealth, which, directly and indirectly, would bring into the treasury a revenue, and this, within a very few years at most, would return to the Treasury the entire original investment of the Government.

And the speaker ventures to say that the more the people of this country study this problem the more they will be convinced that there is no expenditure which the National Government could make which would so enlarge its national resources, and build up everything that goes to make this country great, as the storage by the National Government of the great floods that now go to waste in these western rivers, carrying death and destruction with them. In a few years from to-day it will be hard to find a man who will be willing to confess that he ever stood up and objected to that policy.

The question as to storage reservoirs and the regulation of the floods of great rivers is very fully covered and answered in the Chittenden Report. The whole subject is thoroughly discussed. That report, being the engineering point of view, is by Captain Chittenden, one of the best engineers in the United States Army. The speaker is unable to point to any locality in this country where lands have been reclaimed from overflow by the construction of storage reservoirs built for that purpose. One reason for that has been that, up to the present time, there has been so much land that such reclamation was not required. But with 80 000 000 people in this country to-day, and the

probability that the population will double, and that within 30 years there will be 80 000 000 more, for whom homes will have to be provided, the speaker thinks that the practicability of the proposition will be demonstrated, in the next few years, as it has already been proved theoretically by Captain Chittenden. Mr. Maxwell.

What is known as Tulare Lake, in California, has been practically reclaimed by irrigation, because, for a number of years, the water which formerly flowed into the lake has been diverted by canals from its original channels, and used for irrigation. The bed of the lake is now dry and is good agricultural land. At one time it was a large lake. The canal systems, of course, are not reservoirs, in the exact sense, but they serve the same purpose and have had the same effect upon the lake as though the irrigation systems of San Joaquin Valley had been reservoir systems.

J. JAMES R. CROES, Past-President, Am. Soc. C. E.—Before committing this Society to the advocacy of the undertaking by the United States Government of a scheme of public improvements having the scope and magnitude of the one which has been so forcibly presented by the previous speakers, definite information should be had on a few points which, so far, have not been clearly stated. Mr. Croes.

Is there in existence, anywhere in the world, an extended area of territory which at some past time was subject to devastating freshets, but is now free from injury from this cause, in consequence of the construction of storage reservoirs on the line of the streams?

Is there any well-recorded instance in Europe or America where the construction of impounding reservoirs on the upper waters of the tributaries to a great river has been effective in reducing and regulating the floods on the line of the main stream?

Is there any case known where the freshets on any stream have been prevented or materially reduced by the construction and maintenance of a series of impounding reservoirs along the course of the stream?

In general, is the proposed policy purely theoretical, or has it been tested by actual experience on a considerable scale?

The questions suggested seem to have been sufficiently answered. There is no positive evidence that lands have been reclaimed or freshets prevented along the line of rivers by the means which it is proposed to apply in this case. Until attempts have been made on a moderate scale, and some experience gained as to methods of procedure and results, it will not be expedient or justifiable for this Society to advocate the undertaking by the General Government of works which must be on a very large scale, and which will necessarily be purely experimental, at the outset.

Mr. Haupt. L. M. HAUPT, M. Am. Soc. C. E.—Extending the discussion a little beyond the limits of this country, and going back to the days of Nebuchadnezzar, we find a record of a pool 40 miles square for the purpose of storing the waters of the Upper Euphrates, which was a successful work. It was afterward destroyed by Cyrus, and the country became a wilderness. Also, the pools of the upper Nile, and the lateral basins for irrigation, furnish ample illustrations of the ability to control the waters of the stream. In regard to this discussion, the speaker is in accord with the views already expressed on the subject. The systems in India and in Egypt also are proofs of the statements made by Mr. Newell in regard to the difficulty of maintaining irrigation systems at private expense as commercially profitable. Most of them have proved to be failures, and in very few cases in this country have they been successful. Therefore, from that standpoint, it would seem to become a necessity for the General Government to reclaim the arid land regions. It should be also remembered that the emigrants who go into these new sections are very poor people, and have not the means or the concentration of capital for making public improvements, and, consequently, are unable to do so. Therefore, it becomes a question of public policy and expediency rather than one of ability to carry out those works. Just before the opening of the Northern Pacific Railroad an effort was made by Herman Clarke and others to organize a large syndicate for the improvement of the Upper Yellowstone District, but it failed. In these works, therefore, it would seem to be proper for the Government to take charge, not only for its own sake, and for the control and irrigation of its own public domain, but for the relief of navigation and the storage of a supply for its rivers.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E.—Upon this subject, there is room for considerable discussion. In the speaker's opinion, such work can only be properly executed upon a large scale, which involves one of two methods: First, that by the National Government, as suggested; or, second, by some great trust.

Of the two evils, the speaker prefers the National Government. Although it might not be done as well, the question of politics being involved, at least the general public would have the opportunity of selecting their own physician.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 934.

THE REGULATION OF ENGINEERING PRACTICE BY A CODE OF ETHICS.

An Informal Discussion at the Annual Convention, May 22d and 23d, 1902.

SUBJECT FOR DISCUSSION:

"Should Engineering Practice be regulated by a code of ethics? If so, how can such a code be established?"

By Messrs. GEORGE A. SOPER, J. A. OCKERSON, BENJAMIN M. HARROD, J. JAMES R. CROES, F. W. DALRYMPLE, CHARLES G. DARRACH, W. HILDENBRAND and GEORGE A. SOPER.

GEORGE A. SOPER, Assoc. M. Am. Soc. C. E.—The question as to whether the practice of the members of this Society shall be regulated by a set of rules or standards of action, is not novel. The desirability of having a code of precepts for the guidance and control of careless or indifferent engineers, who may be over-strenuous in seeking employment, or willing to accept services which are incompatible with the dignity of the profession, is a perennial subject of discussion. Mr. Soper.

There are members of this Society who consider that the vocation of civil engineer is less remunerative and desirable than it need be, and that the way for engineers to improve their status is to form themselves into a more homogeneous moral whole. In this era of great industrial development we have daily reminders of the strength that lies in union. Everywhere we see harmonious working agreements

Mr. Soper. based on the community-of-interests idea, the object in all cases being to reduce the inevitable friction which comes from many independent units working at cross purposes in the same field, and to reap the benefits which result from harmonized and co-ordinated action. From the great steel and steamship combinations down to the unions of laboring men, the utility of a close organization, bound by what may be called a code of ethics, is appreciated.

There is nothing new or formidable about the idea of a code of ethics, any more than there is about a list of by-laws of which, in the case of the Society, the code might very possibly become a part. Moral codes were old in the time of Moses, and have been indispensable ever since. George Washington had a reference guide of action, and Franklin, a chart of conduct. In a broad sense, the Constitution of the Union is a code of ethics, as are the various public laws and ordinances under which we live.

In a narrower sense, and the one in which we must use it here, ethics is the subject which treats of those moral acts which, though they cannot make us liable to bodily restraint, are capable of exercising a powerful influence for the good or ill of our fellow man.

It would be the object of a code of ethics to express the united voice of the Society upon such points as the following:

1.—To what extent and in what ways may an engineer advertise his services?

2.—Under what circumstances is the owning and exploitation of patents consistent with a strictly professional practice?

3.—What general principles should govern the conduct of consulting engineers toward other engineers engaged upon work?

4.—Under what circumstances should an engineer pass judgment on the work of his professional brethren?

5.—To what extent should the circumstances of an engineer's private life be allowed to interfere with his professional standing?

6.—Is it practicable or advisable that the Society should attempt to expose quacks?

7.—What acts, if any, should make an engineer liable to the censure of the Society.

If we consider that every man is a man of principle, that is, has definite ideas as to the moral consequences of his acts, and decides upon a doubtful course of action only after referring the matter to his conscience, we will see that he has a code of ethics, or moral standards, whether or not they happen to be written down. In the same way, this Society, as an aggregation of moral and responsible men, has an unwritten code of engineering etiquette. The main principles are known to all. The finer points, the nicer discriminations, are chiefly in the keeping of the older members. The majority of the young men are constantly improving their professional sensibility through expe-

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rience, and rectifying their courses of conduct by the moral pattern set by the great chiefs. Here and there, there are exceptions; men who appear indifferent to their status, or, like raw recruits, are unable to discern, without help, what is expected of them.

The argument for a written code is the argument that the moral teachings of the pillars of the profession should be crystallized into such form as to be available and accessible to all, down to the last recruit. The argument against, expresses the opinion that the recruit should be left to find out for himself what is good and what is bad for him and his fellows.

As the one who has been asked to open this discussion, the speaker cannot announce himself as a warm partisan of either side. He has felt the need of knowledge of the ethics of the profession, and has set out to get it after the usual fashion. He has been an observer of courses of action on the part of some which he thought should not pass without the censure of a united engineering fraternity, and he has seen worthy members of another profession advance through a co-operation and support of their fellows which would be impossible with us under present circumstances.

If there is need of a written code of ethics in the American Society of Civil Engineers, that need should make itself felt naturally and spontaneously. A code, based on less than a practically unanimous vote of approval, would be difficult to introduce, and, in the end, might prove more of an incumbrance than a help. But, backed by a solid sentiment, a code should be a benefit.

It may be well to remind the members of some of the circumstances under which the question of a code of ethics was discussed in the Society some years ago.

The question of adopting a code came up for consideration at the Annual Meeting of the Society, held on January 18th, 1893. It was suggested that a special committee be appointed to consider the propriety of adopting a code, and, if a code was found desirable, to prepare one for the consideration of the Society. The matter was referred to the Board of Direction with the request that a definite recommendation be made by the Board to the Society at its next Annual Convention. The Board considered the matter carefully, and published a collation of arguments for and against the step. Finally, at the Annual Convention held at Chicago, in August, 1893, the Board advised that no committee be appointed to consider the question of a code. The subject was actively discussed by the members present at the convention, and the point was raised that so important a question should be decided by a vote of the whole Society through a letter-ballot. A vote as to whether a letter-ballot should be called for on this subject was decided in the negative.

If the members feel differently to-day than they did in 1893, a prac-

Mr. Soper. ticable course of action seems clear. A motion can be made to the effect that it is the sense of this meeting that a letter-ballot be issued to determine whether a committee should be formed to consider the advisability of drawing up a code of ethics for the Society, said committee to be empowered and charged with the duty of drawing up and submitting to the Society such a code, if their decision is in the affirmative.

It is to be clearly understood that this would not commit the Society to a code of ethics, but would make possible a careful and official study of the subject, the results of which would be placed before the whole Society for its action.

The following is quoted from the *Proceedings*, Am. Soc. C. E., Vol. XIX, January–December, 1893, pages 123–130.

"PROPOSED CODE OF ETHICS.

"At the Annual Meeting, January 18th, 1893, the following resolution was presented:

"'*Resolved*, That a special committee be appointed to collect information and to consider the propriety of the adoption by the Society of a code of ethics for the profession, and to make such other recommendation as the committee thinks proper.'

"This resolution was declared to be subject to the provisions of Section 13, Article VI, of the Constitution, and was, under those provisions, referred to the Board of Direction.

"The duty of the Board in such a case is declared by the Constitution as follows:

"'The Board shall consider the resolution and report its recommendations to the Society at the next general business meeting, together with a statement of the arguments for and against the appointment of such committee.'"

"It is clear that the Constitution (Article VI, Section 13) makes it a little difficult to appoint a special committee, and requires a large approval of the Society to appoint a committee to act on any such subject. The Chair will assume that most of the members have read, or glanced at, the arguments prepared by the Board of Direction. The recommendation of the Board is on the last page.

"Is the meeting ready to take a vote on this question, or are any gentlemen desirous of speaking on the subject which is now before you for consideration?

"J. B. JOHNSON, M. Am. Soc. C. E.—These papers, Mr. President, have just this moment been handed us, and every one has not had time to read them.

"The PRESIDENT.—If it is desirable to discuss the question, the Chair will read over the recommendations, *pro* and *con*, of the Board, and it can be taken up. It should have proper and careful consideration

"Mr. JOHNSON.—Could this matter come up at the next session, Friday evening?

"The PRESIDENT.—No, sir; it is to be settled by a vote, requiring 'a two-thirds vote of all the members present.' That vote must be

taken this evening. Perhaps it will take very little time; it will be just as well to read the recommendations of the Board. Mr. Soper.

"(Read the report as follows):

"'The Board of Direction herewith presents a statement of the arguments for and against the appointment of such committee to the Business Meeting of the Annual Convention.

"'ARGUMENTS FOR THE ADOPTION OF THE PROPOSED RESOLUTION.

"'1. The resolution is for the appointment of a special committee of this Society, to collect information and to consider the propriety of the adoption of a code of ethics, and to make such other recommendation as the committee thinks proper. This does not commit the Society to the adoption of a code of ethics, but simply proposes the appointment of a committee on this general subject. There should be no objection to the appointment of a carefully selected committee to consider the subject, and the members of the Society will certainly have larger information and be in a better position to determine as to the propriety or impropriety of the adoption of a code of ethics after the report of such a committee than without such report, the subject never having been carefully considered or presented as a Society matter.

"'2. The difference of opinion on this general subject which evidently exists among engineers is a good reason for the appointment of the proposed committee, so that the subject may be presented for further consideration and discussion in a clear and definite shape.

"'3. There seems to be some question as to whether civil engineering may properly be called a profession. A clear statement of what the word means is given by the 'Century Dictionary,' which defines a profession to be:

"'The calling or occupation which one professes to understand and to follow; vocation; specifically a vocation in which a professed knowledge of some department of science or learning is used by its practical application to affairs of others, either in advising, guiding or teaching them, or in serving their interest or welfare in the practice of an art founded in it.' The same authority adds: 'Formerly, theology, law and medicine were specifically known as *the professions;* but as the applications of science and learning are extended to other departments of affairs, other vocations also receive the name. The word implies professed attainments in special knowledge, as distinguished from mere skill; a practical dealing with affairs as distinguished from mere study or investigation; and an application of such knowledge to uses for others, as a vocation, as distinguished from its pursuit for one's own purposes. In professions strictly so called a preliminary examination as to qualifications is usually demanded by law or usage and a license or other official authority founded thereon required.'

"'This definition seems very clearly to apply to the actual facts of the practice of civil engineering. Men who have any right to be called civil engineers, and who have any right to be corporate members of this Society, do have a vocation in which a professed knowledge of some department of science or learning is used in practical application to the affairs of others. If the proposed committee finds it desirable it would doubtless recommend some means of carrying out or of enforcing regulations for such of the relations of engineers to their clients and to each other as could at all be made subject to rules. At all events, it is desirable that the committee should study this subject and report to the Society.

Mr. Soper. "'4. Rules do exist which govern the relations, to each other and to the public, of men following other professions or vocations in which there are associations of the members of such profession or vocation in some respect similar to this Society. This applies, not only to the old so-called professions, but to other vocations. The dealers in stocks are subject to clear and definite rules, formulated and enforced through their exchanges. The dealers in produce are also subject to rules through their exchanges, and this is the case in other vocations.

"'5. It will be well to appoint a committee to consider whether the dignity of civil engineering and the relations of members to themselves and to the public might not be improved by the application of rules of ethics or of conduct to be adopted by this Society, as a representative body.

"'6. It seems to be a fact that civil engineers do permit themselves in some few instances to act in ways which in other professions or vocations would be generally considered improper and unprofessional acts. Such acts are contrary to the rules of etiquette which have become standard in those professions. The appointment of a committee to see whether this can in any way be remedied as regards civil engineers is desirable.

"'7. The lack of rules by which departures from the proper line and professional line of conduct can be judged affords excuses for unprofessional conduct when such conduct is based upon ignorance of, or indifference to, professional morals. The adoption of rules and principles governing the relations of engineers to each other and to their clients would restrain the unprincipled and guide the ignorant.

"'8. The fact that many engineers are engaged in salaried positions should be an argument in favor of, rather than against, the appointment of a committee to consider the possibility of a code which would add to the dignity of the profession. Those who require the services of engineers are apt to base their appreciation and estimate the value of those services upon the standing which the engineer himself takes. It is inconsistent with the function of this Society to depreciate engineering. It is its duty to endeavor to elevate it in every relation, and the appointment of the proposed committee will be a step in this direction.

"'ARGUMENTS AGAINST.

"'It is held that one may just as well assert that the other great professions are great in spite of their codes as because of them. Neither assertion can be proved. It seems to be a fact, however, that the lawyers have no formal code. Their rules of conduct are partly matters of tradition and partly of interpretation and construction, by various authoritative bodies, and form a valuable mass of professional ethics, but they have not been codified and adopted by the great Bar associations in the United States.

"'Furthermore, one of the most important medical societies of the United States, the Medical Society of the State of New York, has voted to abandon its code of ethics. The fundamental reason for this action is summed up in these words of the president of that society: "It would comport more with the dignity of the medical profession, and would enhance the respect in which it is held by the general public, if all specific rules of ethical conduct were elided from the by-laws of the State Medical Society, and if the regulation of such matters were hereafter left to the judgment of individual practitioners influ-

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enced by the well-known consensus of professional opinion and by local custom."

"'It is believed that the architects also are without a formulated and accepted code of ethics. So we see that the professions are either without codes or are beginning to abandon them. The rules governing the transactions of the stock and produce exchanges are not analogous to what is understood by a code of professional ethics; they provide simply for the carrying out of specific contracts, many of which are not enforcible by law, but the carrying out of which is vital to the existence of the business of the exchanges.

"'It is held that it is impossible to frame a code of ethics that would cover all cases. Such a code would be too complicated and minute to be successfully administered. The code, therefore, must be merely a statement of general principles; but such statements have been made by moral teachers ever since society began, and the general principles which govern human conduct are sufficiently well formulated already.

"'A strict code would restrain those who need no restraint, but it would not be regarded by the unprincipled further than their own interests dictated. It would happen sometimes, perhaps often, that an engineer would be deterred from doing his duty to his client or to himself by a timid obedience to the code, or that he would make the code an excuse for not doing disagreeable things which reflected upon the honor or capacity of other engineers.

"'It is held that specific cases of violation of the traditions and spirit of the profession should be treated individually, each on its merits, and that this can be done, so far as the American Society of Civil Engineers is concerned, by presenting the case to the Board of Direction, or that a special body or committee can be created in the Society to perform this special function.

"'The Board of Direction recommends to the Society that no such committee be appointed.

"'By order of the Board.

"'F. Collingwood,

"'*Secretary.*'"

"The President.—The subject is now before you for discussion; what shall be done?

"Mansfield Merriman, M. Am. Soc. C. E.—As I understand the matter, the resolution that a special committee be appointed was presented at the Annual Meeting of the Society in January. I also understand that this business meeting has no power to act upon that resolution, that such a committee can only be appointed, that such a resolution can only be adopted, by the result of a letter ballot sent out to all the members of the Society, and that the question really before this meeting is whether such a letter ballot should be issued or not. Is that the question?

"The President.—The Chair will read the clause of the Constitution again, for the information of the members.

"(Read 'The Board of Direction shall,' etc.)

"The gentleman will see that the business is in the hands of this meeting and must be disposed of, under the Constitution.

"Mr. Merriman.—I think I am correct, then, in the idea that the whole question before this meeting is, shall the letter ballot be issued?

Mr. Soper. "MENDES COHEN, Past President, Am. Soc. C. E.—The only motion that is proper to be offered, under the Constitution, is a motion from some corporate member present that such letter ballot shall be issued. If there be no such motion made, the matter rests just where it is now; if there be such a motion made, under the Constitution, the question then comes before the Society in this meeting, and individual arguments can be heard for and against it, and then a vote upon the question must find two-thirds of the corporate members present in the affirmative before such action can be taken. It is only if somebody here will make the motion.

"The PRESIDENT.—That statement is perfectly correct and in accord with what the Chair stated.

"Mr. MERRIMAN.—I move that it is the sense of this meeting that a letter ballot shall be issued. (Seconded.)

"The PRESIDENT.—It is moved and seconded that it is the sense of this meeting that a letter ballot shall be issued on the subject; the matter is now before the members for discussion.

"G. KAUFMAN, M. Am. Soc. C. E.—As this is a question of interest to all members of the Society, I don't think that a small meeting like this should vote this motion down; it would be better to have all the members vote upon this question.

"JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—Mr. Chairman, the arguments against are directed simply against the propriety of a code of ethics, not the appointment of a committee. The first argument in favor of the resolution points out that all that is asked is the appointment of the committee. The arguments against seem to be directed against the adoption of a code, not the appointment of a committee.

"The PRESIDENT.—There certainly could be no objection to the appointment of a committee if there were no objection to the work the committee was to do. Are there any further remarks to be offered on the motion?

"ROBERT MOORE, M. Am. Soc. C. E.—Mr. President, I wish to express my hope that the Society will not take any steps looking to legislation of this kind. I think that the conduct of members of the Society can fairly be left to their intelligence and their intention to do right, and I do not think that we should be put in the leading strings of legislation of this Society in that matter. I think the arguments in the negative of this question are sound, and I hope that no steps will be taken to inaugurate a hard and fast code of ethics such as seems to be contemplated by the proposition.

"Mr. COHEN.—I would add but one word to what has already been said upon the subject. It appears to me that these codes of ethics, of which we hear so much and see so little—if you inquire you can scarcely find a written code of ethics, we do not see it in other associations—that a code of ethics must be governed by and formulated upon the good sense, good breeding and general comity of man to man. Now, as has been stated in this argument, such government will obtain—it has obtained and will obtain—among men of experience in the profession, men who are trained, men who are educated; and when, here and there, you will find an exception, some erratic course will be taken, it will be much easier to regulate that within the Society if an extreme case arise, than to refer it to book. Ten to one there will be no case provided in the book, and then you have to go back to the Society in the end.

"The only objection to the appointment of a committee to consider such a question is that it is dangerous to the Society to present to the

committee such an argument and stir up a matter within the Society which, at best, can do no good to the Society and which may result in injury. I have no idea myself that even if such committee be appointed and it discuss the subject, and a letter ballot be issued, I have no idea that the Society would deliberately adopt such a course. If the view that I hold, and that I think a great many hold, is correct, then for the Society to adopt such a thing would be dangerous; it is a great deal better that it be not adopted and not considered. Mr. Soper.

"A. Fteley, M. Am. Soc. C. E.—As an engineer of 25 years' standing in this country, I have been called upon, not only to perform regular duties, but I have very often been called to consult, alone or with others, on matters of importance; I have therefore had a great many experiences of various kinds. I have found in my case and in the case of others that a great many occasions arose in which the conduct of engineers was questioned. I have known of such cases where there was not the slightest reason for it. Those questions will arise, especially as to the relations of engineers and contractors; we know how much antagonism may arise. I have been a member of the Board a great many times. I have seen cases where the reception of a candidate into the Society has been fought by means fair and unfair. Sitting in the Board, I have seen our Secretary presenting to the Board stacks of letters against the reception of members simply because they have done certain things. Nine times out of ten we have found that such statements were made by interested parties who had an axe to grind, or who were taking their revenge upon somebody. It seems to me that if we submit a case like this to a code of ethics, in many cases the book will say nothing. We have duties to perform, and I suppose a great many of us try to perform them as well as we can. It seems to me that in cases of that kind a man must fearlessly follow the line of duty he believes he has to follow and let him stand by it or fall by it. It may happen once in a long while that a man may suffer unjustly, but in 99 cases out of 100 the man who is worthy, even though he has been attacked, will in the end triumph because he is a good man.

"I am very proud of our profession. I know one thing by my experience in public works, and it is this: If an action has to be passed by certain authorities outside of engineering, I find that there must be the signature of this one and of that one before action can be taken; while we engineers generally put our signatures to very large sums, and it means simply that the money will be paid without question. Consequently, why should we submit engineers who have such a high standing in the community to rules which will simply narrow the limits within which we are working. I believe it is an injustice; I believe it is an injury to our profession to try to bring us within such limits as this resolution contemplates.

"The professions of the law, of theology, of medicine, have been mentioned. The gentlemen who belong to these professions, before they can practice, have to be accepted by certain authorities. We are not in the line of these conditions; we have a great many worthy men who have simply put their shingles out, and, in many cases, have made a success of it. Our profession cannot be compared to other professions. So far as I am concerned, I believe that this resolution is one that is likely to disturb the even tenor of our ways, and I emphatically say that we ought not to consider the idea of having a committee appointed for the purpose mentioned. (Question called for.)

"The President.—Corporate members only can vote. Understand, the question is, shall a letter ballot issue, or shall it not? A vote in

Mr. Soper. the affirmative means that the Board shall ask the Society to vote on the appointment of a committee. A vote in the negative means that the whole matter is dropped here by the Convention. Those in favor of the issue of a letter ballot will please rise to their feet. (11 arose.)

"All opposed will please rise to their feet. (54 arose.)

"Eleven in the affirmative; 54 in the negative; the motion is lost."

Mr. Ockerson. J. A. OCKERSON, M. Am. Soc. C. E.—The agitation in engineering societies and in the technical press as to the propriety of a code of ethics to govern and guide the members of our profession may be taken as evidence of a desire to ennoble the calling of the engineer. It is a recognition of the fact that engineers, in this country at least, are not yet accorded the position to which they are entitled by the importance of their work and the conspicuous part they have taken in the development of the natural resources of our country. There are, however, some signs of progress in this direction.

None of the so-called learned professions is so intimately connected with the material progress of our country as is that of engineering. The engineer has always been the advance guard of civilization in the development of our western empire, and, largely through his instrumentality, a marvelous chauge has been wrought in a few decades, which would, ordinarily, be the work of centuries.

In the older and more settled portions of the country his work is equally important, in providing ample means of transportation by land and by water, and in many ways contributing to the comfort and safety of the people who live in our great cities.

Yet, in the face of all this, the profession is not held in that high public esteem to which it is justly entitled. This is due, in a great measure, to the fact that the public is not as familiar as it should be with the functions of the engineer, whose individuality is more or less overshadowed by the great corporation which employs him.

The education of the public up to the proper appreciation of the services of the engineer is necessary, before any great improvement in his status can be hoped for. It can never come through the establishment of any possible code of ethics. No opportunity should be neglected which will serve to bring the engineer and the public to a better understanding of one another. The engineer, on his part, should cultivate the acquaintance of men in other walks of life.

Much can be done at great expositions, by bringing the general public into contact with the work which the engineer has wrought and which emphasizes his science, ingenuity and skill.

Heretofore, engineering work in expositions has been somewhat obscured by being mixed up with transportation and other related matters which indeed are closely akin to it.

In the coming exposition at St. Louis, civil, military and architectural engineering and engineering pertaining to public works, will be housed together in the Liberal Arts Palace. Mr. Ockerson.

In the interests of the profession, as well as that of the exposition, and through your aid, the speaker hopes to secure for the several groups devoted to engineering the best examples of what the engineer has wrought in all parts of the world. With the co-operation of the engineering profession, which should be readily accorded, there will be gathered at the coming exposition a display which will do credit to the profession, and which will impress the layman with the dignity and importance of our calling. Every engineer can do something toward the realization of this much-desired result.

The speaker will be very glad if each member will take this up as a personal matter, and arrange to offer models, plans, drawings and photographs of some of his important work. It would also be well to have information as to any important engineering works which would be interesting and novel as exhibits.

BENJAMIN M. HARROD, Past-President, Am. Soc. C. E.—The speaker has but little to add to this discussion beyond the expression of great pleasure in hearing that part of the Annual Address* devoted to the subject, and of entire concurrence in the views expressed by the President and by the Committee on Regulating the Practice of Engineering.† Mr. Harrod.

Effective means already exist by which any desirable improvement of the relations of civil engineers to each other and to the public can be accomplished. The principles of professional ethics should be explained and impressed in engineering schools, and a full and clear discussion of them should form part of the Transactions of Engineers' Societies.

In this way an unwritten law will prevail, with all the authority, and more completeness and flexibility than can be given to a written code.

J. JAMES R. CROES, Past-President, Am. Soc. C. E.—To the first question propounded, "Should Engineering Practice be regulated by a code of ethics," an affirmative answer must be given. No business, trade or profession can be successfully carried on in any community except in accordance with the ethical standards which prevail in that community. Such standards have varied, at different stages of the world's progress, and among different nations, mainly in accordance with the different religious concepts which have prevailed in certain periods and among certain communities. Among what we consider the civilized nations of the present day, the standard of ethics is, as has been well said, embodied in the Ten Commandments of the Mosaic Mr. Croes.

* *Transactions*, Am. Soc. C. E., Vol. xlviii, p. 227.

† *Proceedings*, Am. Soc. C. E., Vol. xxviii, p. 188.

Mr. Croes. Dispensation and the Golden Rule of the Christian Dispensation. The application of these fundamental principles to special cases, and the interpretation of them, are matters of tradition, experience, equity and jurisprudence.

The second question propounded is: How can such a code be formulated and established? The answer to this must be that, under existing circumstances, the framing of a written law of action applicable to civil engineers alone, is impracticable.

The formulation of a code of rules which would be applicable to all cases in which doubt might arise in the mind of a practitioner of civil engineering as to what was the proper course for him to pursue, would involve the consideration and adjudication of almost every conceivable case of "conscience." Supposing, however, only a few general principles to be enunciated, those principles would have to be accepted by the persons for whose guidance they were intended, and an obligation entered into by such persons that they would be liable to some kind of penalty in case of their infringement. This would require the establishment of a court of arbitration or adjudication. Any attempt to establish such a code and such a court, by a society or association representing only a small proportion of the practitioners of the multifarious occupations embraced in the general title of Civil Engineering, could only result in ridicule and failure.

Irrespective of the outsiders composing the great mass of the profession, it is very questionable whether unanimity could be secured in the organization undertaking to frame a code. Our own efforts to frame a Constitution adapted to our needs furnish an instructive lesson in this respect. For years not a general meeting has been held at which changes or modifications or explanatory additions to our fundamental law have not been strenuously demanded by some one who was dissatisfied with or could not comprehend a clause or a provision of the Constitution. How much greater confusion of ideas might be expected if ethical and abstract questions were at issue!

But, while the promulgation of a formal code of ethics for the Civil Engineer is not practicable or desirable, there is doubtless a need and a demand for a public setting forth of some of the elementary principles which have come to be recognized by engineers of experience in the combined management of materials and men as fundamental, but which the young practitioner, who, fresh from his college halls, attempts to grapple with practical problems of design, construction and management, has no familiarity with, and of which the ordinary man of business, unfamiliar with professional ethics, has little conception. The young engineer can only imbibe loyalty to those principles by association with older members of the profession, and, lacking the opportunity and too frequently the desire for

such prolonged association, is apt to go astray and commit some solecism in morals, through ignorance rather than viciousness. It would be an advantage, therefore, if some civil engineer of acknowledged experience and standing would write a brief compendium of engineering ethics, from his point of view, and have it published by a standard bookseller. It would promote discussion and lead to much good. But, as has before been said, any effort to give to such a publication the status of an established law of morals would be absurd. Mr. Croes

A book of this kind might also serve the purpose of deterring clients from making to professional men propositions which may be considered either as silly or criminal, according to the point of view. There would have to be laid down some rules so rudimentary as at first glance to seem absurd. The principle, for instance, that no man can serve two masters, seems so self-evident to anyone who has been trained in the doctrines of the Talmud or the Bible, as not to need enforcement. And yet only a short time ago in a seaboard city, where a new administration, pledged to economy, had been installed and had reduced the compensation of all its professional advisers, a number of persons, not connected with the government, and desirous of having their ideas of the administration of an important department carried out, combined in an offer to pay, to a professional adviser whom they selected, an additional salary equal to that which the city would pay him, on condition of his appointment to the position. In other words, they proposed that this city official should have two masters; one, the city whose interests he was supposed to serve, the other, a half-master, with no responsibility to the taxpayers or anyone else except themselves, but whose interests he would be compelled to serve or lose his salary. And yet this extraordinary proposition, based apparently on a kind of electromotive idea of having one machine and two power-houses in different places and under separate control, was made in good faith by men who could not see that it was scandalous and abhorrent to all persons who had correct ideas regarding professional duties and responsibility, and that, moreover, the same rule might be carried out with regard to any other Association, or Club, or Hall, which might be willing to "put up the stuff" to have a city department run as it wanted it.

A rudimentary treatise on ethics might then be a good thing to have, but not as a formal declaration by this Society of what a civil engineer ought or ought not to do, to be, or to suffer.

F. W. Dalrymple, Assoc. M. Am. Soc. C. E.—The speaker hardly understands whether it is proposed to create an engineering trust or an engineering labor union, but it seems to him that either proposition would be beneath the dignity of the profession. If a code of rules is established, it would certainly be necessary to create at least some Mr. Dalrymple.

Mr. Dalrymple. sort of a court to administer and interpret those rules, which is thought to be undesirable and impractical.

The speaker begs to suggest that, if such a code is to be adopted, the best that could be formulated would be the Ten Commandments, with the addition of that other biblical proposition, to do unto others as you would have them do unto you.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E.—A code of professional ethics may be expressed briefly in three words, known, or supposed to be known, and acknowledged, by any and everyone entitled to a membership in the various grades of this Society. Unfortunately, in the speaker's experience, and probably in that of many other members of this Society, ignorance of these words alone raises the question at issue; and, that the speaker may not be misunderstood, it might be well to make reference to where this rule may be found.

In an ancient compendium, which, by some singular fortuity, has been reproduced in manuscript and the press for nearly 3000 years, and is supposed to be in general circulation, one of the earliest writers, who, if the speaker's memory serves him right, was born in Egypt, enunciated the general principles. Of this incidental reference, the speaker thinks it is unnecessary to make more specific mention.

The difficulty seems to be in the application of the rule. In other words, to make the punishment fit the crime, or, probably it might be more aptly stated, to make the reward fit the merit. It would seem to the speaker that the first duty of a member of this Society, regardless of his experience and scientific attainments, is to be a gentleman, with that regard for each other member of the Society, at least, that he would expect awarded to himself. Unfortunately, this condition does not prevail. The speaker has known of cases where members of this Society have filched work from their fellow-members, have spoken in disparaging terms both of their business and scientific attainments, and, in fact, have done all that they possibly could to degrade the profession into a trade.

There was a time when to be a Member of the American Society of Civil Engineers meant more than it does to-day. In the speaker's experience, this, to a certain extent, is due to the present method of elections, or the practical exclusion of the blackball. The speaker remembers some years ago, and he thinks it was about the time of the first St. Louis meeting, that a certain person received some seven or eight blackballs. A great hue and cry was raised that a conspiracy had been raised against the individual in question. The speaker cast one of those blackballs, and used no influence whatever in the casting of the others. The candidate under discussion had been given a position by the speaker, to a certain extent out of charity. The aforesaid candidate had hardly been warm in his position when, like Ephraim, he "waxed fat and kicked." The chief engineer, whose principal

assistant the speaker happened to be, apprised him one day of the fact that the speaker's protégé was earnestly endeavoring to obtain the speaker's position, without even having the courtesy of mentioning the matter to him. Mr. Darrach.

No doubt many of our members could recite parallel instances, and it is exceedingly difficult to be obliged to practically convict a man of a heinous offence, which the present laws of the Society practically require, before he is excluded from the privilege of membership; but we are in the situation and must enact certain laws or bring attention to the various members who are not familiar with the ancient book to which the speaker has referred.

Mr. Hildenbrand.

W. Hildenbrand, M. Am. Soc. C. E. (by letter).—The fact that the question of establishing a code of ethics for practicing engineers is to be discussed is in itself an indication that past experience must have shown that the conduct of some members of the profession has not always been what it should. This is not surprising, because human nature is the same everywhere and at all times; knowledge and mechanical skill do not necessarily exclude selfishness and injustice, which are found in all kinds of business and in every station of life. If it were possible to eradicate from human society the two evils of selfishness and injustice by a printed code of ethics, it would be our duty to adopt one, without a minute's delay. Such a code could be short and very simple; it could be comprised in a single sentence, like this: "Act as a gentleman, in every respect, on all occasions; be just and generous to your brothers in the profession, and do not hesitate to take the full responsibility for all your actions!" These maxims, of course, are not new; they have always existed as unwritten laws; they fit every profession and vocation, and if they were inherent to all men and had always been fulfilled, or would be fulfilled, no special code of ethics for engineers would be necessary. Unfortunately, there is no such perfection, and men have always existed who either lack the natural capacity of making a fine distinction between right and wrong, or whose better judgment and better sentiments are darkened by selfishness. The writer fears that no code of ethics would rectify the evils done by such men, because, if it is not in their nature to act justly and generously, no amount of printed regulations will be able to change their character. Moreover, it would be almost impossible to enumerate and define beforehand in special paragraphs the hundred and more varieties of forms of injustice. Certain actions may be justified in one case and condemnable in another. It might happen that a moral or professional injustice might be done, one that is not defined in some paragraph of the code, and the transgressor, though guilty, would have to be acquitted for not having violated the printed regulations of the code of ethics. By ethics is meant the doctrine of moral duties, and not

Mr. Hildenbrand.

a mere code of etiquette as to superficial outside forms of conduct on certain occasions.

While the writer thinks that it might be desirable to establish some simple forms of etiquette, not as absolutely necessary rules, but as advice or for convenient guidance, to the novice in the profession, he believes decidedly that a code of ethics for regulating engineering practice is not only useless, but, inasmuch as it can never be complete, it will do more harm than good. The true ethics are, or should be, implanted by Nature in every man; but, outside of that, the writer believes in letting everybody exercise in the broadest sense his full liberty, and not be hampered with regulations, as long as he does not interfere with the rights of another.

For determining whether any rights have been infringed, and for rectifying any wrongs done, the writer would propose, in place of a code of ethics, the establishment of a court of justice as a part of this Society. How to arrange the details of such a court of justice is a matter to be discussed afterward, but, in order to accomplish the desired ends, the court ought to be permanent; that is, it ought to constitute a special part or committee of this Society, having nothing else to do but examine and judge upon the disagreements of ethics among the members of the profession. The principal quality of this court, of course, should be its impartiality. Every member of the Society, whether chief engineer of a department or only a humble draughtsman, should be equal before the court, and have the right to present to it his real or supposed grievances. The court would examine these and give a verdict accordingly.

As it is at present, there is absolutely no redress for a member of the profession who suffers injustice of a kind which does not come under the regular civil laws. To mention only one instance: It happens, not infrequently, that an engineer is deprived of the credit due him for his work, in consequence of certain publications, in which, knowingly or unknowingly, the author attributes the merit to another person who does not deserve it. What can one do in such a case? Should he write to the papers, or publish another book on the same subject, merely for contradicting a certain statement? As a rule, this is impractical, and also inefficient; but let the matter be brought before a court of justice of engineers, who will make an official report, to the profession, of the true state of affairs, and who has the power to mete out punishment by reprimanding the transgressor, or, in severe cases, by his eviction from the Society. It would give general satisfaction, and would do more toward establishing a true code of ethics among engineers than specifying and printing a number of arbitrary or self-evident rules.

The writer does not imagine that such a court would eradicate all injustice among members of the profession, as little as the ordinary

courts are able to abolish transgressions against the civil laws, but he believes it would have a wholesome effect in reducing the violations of ethics, and in rectifying, to a great extent, the evils arising from such violations. This Society is large and powerful enough to make its court of justice generally respected. Mr. Hildenbrand.

The object of this discussion proves that the need for rectifying or bettering the ethics in the engineering profession has been felt, and the writer believes that such a need cannot be better or more efficaciously supplied than by a body of truth-loving men known as: "The Court of Justice of the American Society of Civil Engineers!"

George A. Soper, Assoc M. Am. Soc. C. E. (by letter).—In declining the opportunity which the opening of this discussion presented to launch an agitation for a code of ethics, the writer believes he has acted wisely. Nothing short of a unanimous demand would have produced a code that would have been desirable or useful. By allowing the matter to rest where it has so long lain, the Society has avoided what might have been a fruitless, protracted and undesirable discussion. The verdict is against a code. More than that; impliedly, no full discussion of the topic is wanted. Mr. Soper.

There are few arguments which, if once fairly started in this Society, would be likely to arouse a more extended discussion than the question of a code of ethics. Nearly every member has some views on the subject, and some evidently hold firmly rooted convictions as to the need or expediency of a code of morals for engineers by which to govern their practice.

The value of such a discussion would be doubtful. We have seen how the medical profession has been divided on the question of a code of ethics, and how long and how hopeless have been the efforts of physicians to reconcile the opinions of those who do and those who do not favor the adoption of a code.

When it is remembered that most men hold to their opinions on religious and moral subjects with peculiar tenacity, and are by nature jealous and sensitive of their honor, it does not seem strange that many should look upon a code of ethics as upon a moral straight-jacket into which they decline to enter without warrant.

And, on careful thought, the writer concludes that there is no warrant for a code of ethics in the American Society of Civil Engineers. The members are chosen from among the most respectable and respected members of their calling, and the machinery of election is designed to exclude from membership all who have exhibited, or seem likely to exhibit, anything less than perfect moral fitness. Receiving their franchise from fellow engineers of high standing, the very badge of the Society worn by members is a mark of probity and honor. And, let it be said, few who wear that badge fall from grace.

Under present circumstances, such rare cases of moral obliquity as

Mr. Soper. occur among civil engineers find, and should find, a proper rebuke in the example of rectitude set by the majority. In these examples, and in the literature of many societies, lies ample instruction for those who are minded to get it.

A potent argument against a code is, that since it is impossible to frame rules which will cover every possible case in which the question of moral conduct is involved, a code must be at best but a provisional and imperfect, and hence only a measurably successful, affair. Moral questions can seldom be settled on general principles. Cases differ. Individual circumstances are often very important in modifying a judgment. Notwithstanding the great care given to the perfection of the penal code, justice at law depends as much upon the ability of judge or jury to fit the punishment to the crime as upon the law itself.

And a code of ethics, like all formal provisions of law, would only be a corrective or preventive of injustice, and not an important educational instrument making for the material and moral advancement of engineers. Material and moral advancement cannot be forced; they must be cultivated. They can best be cultivated in the schools, in the technical societies, at home, and in the daily routine of work.

A code of ethics, by specifying some lines of proper and improper action, would throw into recognizable form the bearings of some courses of conduct which might be overlooked by some, but the effect which it would have upon the character of such persons is at best doubtful.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 935.

UNIT COSTS OF WORK IN PROGRESS.

An Informal Discussion at the Annual Convention, May 21st, 1902.

SUBJECT FOR DISCUSSION.

"Is it possible and desirable to keep accounts of cost of work in such a manner as to ascertain unit costs on each class of work?"

BY MESSRS. S. WHINERY, CHARLES S. CHURCHILL, T. CHALKLEY HATTON, OBERLIN SMITH, CHARLES WORTHINGTON, FOSTER CROWELL, CHARLES G. DARRACH, SANFORD E. THOMPSON, W. W. CUMMINGS AND S. WHINERY.

S. WHINERY, M. Am. Soc. C. E. (by letter).—Two questions are embraced in this topic: The first relates to the possibility and the second to the desirability of account-keeping intended to disclose the unit cost of work. Mr. Whinery.

The first of these questions has been practically answered by actual experience. Except possibly in some cases where abnormal conditions prevail, it is entirely possible to keep accounts which will show correctly the unit cost of any work, whether that work be in the nature of surveys or other preparatory operations, or of the construction of engineering structures, or of manufacturing. Such accounts are now, not only common, but almost universal, in all well managed enterprises.

They differ in scope, detail and accuracy, it is true, but the cases are very few where something of this kind is not attempted and

Mr. Whinery. carried out with more or less success, among modern enterprising business men.

Like all other forms of accounting, the value of the results attained will depend upon the adequateness of the general plan and details of the scheme of accounts adopted, the fidelity and accuracy with which the original records are made, and the ability and skill with which these records are compiled and deductions made therefrom.

Each of these three subdivisions of the work is of equal importance, and if either is defective, or neglected, the results attained will be unsatisfactory or entirely worthless. To devise a scheme for a cost-keeping account, with all its necessary details, and to prepare full instructions for its operation, is by no means a simple thing, requiring as it does a very intimate knowledge of the method of conducting the particular kind of work on hand. The difficulty increases with the complexity of the work and the number of unit-cost items it is considered necessary to ascertain. It is only after one has had an extensive experience on similar work that one can frame a scheme that will be workable and efficient. It must be simple and easily understood; otherwise the employees who must be depended upon to make the original records will get them hopelessly confused; or a disproportionate amount of time and labor will be consumed in making them. It must be explicit in its details and instructions, for the same reason, and it should be so comprehensive as to disclose where every item of money or labor has been expended. But, however perfect a cost-keeping scheme may be devised, it will prove worse than useless if not intelligently and implicitly adhered to by those whose duty it is to make the original entries or records. Every cent expended upon the work, whether in the form of cash or labor, must be accounted for and charged to its appropriate item. This sounds simple enough, but only those who have had exasperating experience with the stupid or negligent employee can appreciate how difficult it is to have these original records properly made. Assuming a good system of accounts and accurate original records, their value will depend upon the care and fidelity with which the original information is compiled and made available for use. In the press of other office duties this part of the work is often neglected or postponed until such a mass of it accumulates that it is then abandoned, or it gets into such a state of confusion that it is difficult or impossible to disentangle and tabulate it, and it is then thrown aside as worthless, with the statement that the problem of determining unit cost is hopeless.

It may be confidently asserted that, with the same amount of care and accounting skill necessary and usual in a first-class mercantile establishment, it is possible to determine with accuracy the unit cost of any engineering work.

A knowledge of the unit cost of work is of value in many ways, and accounts which enable us to ascertain it are therefore desirable. To the engineer engaged in designing modern work the important problem is how to accomplish the object sought with the least expenditure of money. Other things being equal, that particular detail will be used which is the cheapest. In order to determine which is the cheaper of two details the unit cost of each must be known. The engineer, therefore, finds it important to accumulate the largest possible fund of cost prices. To the contractor or the manufacturer the question of desirability is synonymous with the question "will it pay," and among the more enterprising and intelligent there can be but one answer. The question of success or failure in these times of sharp competition and low prices may depend upon the accuracy with which cost has been estimated, and this again will depend upon the knowledge of unit cost under the conditions that are to be met. These conditions vary greatly upon different pieces of work, and in different localities, and the estimator needs the accumulated knowledge that comes from habitual cost-keeping. It is important, therefore, that this branch of accounting should never be neglected on engineering work. Furthermore, it is important during the progress of any extensive work. The contractor or the manufacturer who has before him at the end of each week the unit cost of his work during that week, can know whether it is being done with proper economy, and, if it is not, he can ascertain the cause and apply the remedy at once, and thus prevent accumulated losses. Accurate accounts will often disclose defective management which may not be detected by personal observation, during the progress of work. Mr. Whinery.

A careful study and analysis of unit cost will enable those interested to devise improvements and apply economies that would otherwise not be thought of.

There can be but little doubt that the notable success of American manufacturers in competing with those of foreign countries is the result of careful attention to the unit cost of details, and has been made possible by the careful and accurate cost-keeping accounts now found in every enterprising shop.

Both questions embraced in the topic, therefore, may be answered in the affirmative.

Charles S. Churchill, M. Am. Soc. C. E.—Inasmuch as Engineering is the power of producing the greatest and most permanent results for the least ultimate cost, even the intimation that actual and detailed costs cannot be determined and kept is a stroke against the foundation of the profession. Mr. Churchill.

The keeping of and knowledge of these detailed costs and ultimate values really constitute the greatest care of the successful engineer, and the proper use of this information is of almost equal importance.

Mr. Churchill. To be told that a certain piece of first-class ashlar masonry cost $6.50 per cubic yard may attract attention; but this fact is not of much value unless we know, further, how this cost is made up, as it consists of the price of the stone delivered at $2.35, and labor on the same, with other materials, aggregating $4.15.

But we are still not at the bottom, because the cost of cutting different stones varies, and the other materials also vary in cost. Therefore our information is not complete until we know that the amount, $4.15, is made up of cutting, $1.14; laying, $2.26; cement, 20 cents; sand, 7 cents; handling, 16 cents; and setting derricks, etc., 32 cents; and, finally, for comparison, it is necessary to know that on this particular work which has been cited the foreman mason received 30 cents, the regular masons 25 cents, and the helpers 15 cents, per hour.

Considerable care is required to keep and compile such figures; but they are of the utmost importance. For example, the cost of cutting a given stone, together with its delivered price, determines whether or not it can be used in a certain work as against stone from a different locality.

Every contracting engineer keeps track of such detailed costs in his own lines.

The great increase in the earning power of railroad properties in this country is the subject of wide comment. On any given railroad this increase of earning power began with the carefully-planned improvements in grades, alignment and strength of roadway, properly executed by the engineer. It was further secured by having the improvements completed at the lowest ultimate cost; and, finally, by the engineer providing many facilities for keeping down all items of maintenance cost.

Referring now particularly to this last factor, we find that a spirit of rivalry exists in every good workman, and this may be brought into practical use, even if the employee be no higher than a track laborer. It is useless to depend wholly, for the best results, on driving such laborers and their foreman; and the same statement applies to every man worthy of employment on a railroad.

On some closely-operated railroads this plan is followed in much detail and with great success. For example, a simple statement is issued monthly by such a railroad, showing the actual cost of such hum-drum work as the putting of a tie in track on each roadmaster's division of the road. A rivalry is at once induced among all the foremen of every division to be the lowest. A similar statement giving the cost of placing lumber in bridges per thousand feet, B. M., in making repairs, brings about the same rivalry among carpenter forces; and all this produces an absolute and proper reduction in cost of maintenance.

The grand result sought is secured when the total cost of handling

freight per ton per mile is less than during the corresponding month of a previous year. Mr. Churchill.

Comparative monthly statements, in much detail, showing the cost of operation in the different departments of such a railroad, are responsible for many of the large savings secured by the encouragement of the spirit of rivalry among its employees.

It is the engineer on a successfully operated railroad to-day, in point of cost, who compiles much of the information as to costs necessary to secure this grand result in railroad operation.

Surely the engineer's strength and usefulness are largely in ascertaining and keeping unit costs on each class of work with which he is connected, or for which he is responsible.

T. Chalkley Hatton, M. Am. Soc. C. E.—There would seem to be only one side to this question: How can any engineer give his client a reliable approximate estimate of the cost of a proposed piece of work, unless he has first taken very careful estimates of the cost of preceding and similar work? Mr. Churchill mentions the results of taking careful estimates of costs in relation to railroad maintenance and railroad construction, and the care with which such records have been made. The same results have been obtained by the speaker with relation to public work. Mr. Hatton.

Some twelve or thirteen years ago he designed a system of sewers to cost about $1 750 000, and to be constructed from year to year as the funds might become available. When the work began, no reliable approximate costs of the work could be made, as it was to be done under all kinds of conditions; but, at much expense, a systematic record was kept of the actual cost of excavating each linear foot of trench under each condition, whether in rock or wet ground, and whether in paved and unpaved streets; the actual cost of brickwork per linear foot, for each size of sewer and each foot of pipe; and the cost of each manhole, inlet and every other appurtenance, until, at the end of three years, the speaker had compiled a record of actual costs which enabled him to determine, within 1 or 2%, the probable cost of the remainder of the work. These records were compiled in such a manner as to enable him to determine how many bricks could and should be laid by a bricklayer under the several conditions; how many feet of pipe of the various sizes should be laid per day, and the quantity of cement and hemp required to lay it; how much trench one man should excavate in a day, and how much per cubic yard it cost to remove rock.

At the end of three years it was found that the contractors were making too much profit, and the speaker recommended that the minor portion of the system be built by day's labor, and that the larger work, requiring special plant, should be done by contract. This work proved so successful that it was determined to do all work by

Mr. Hatton. day's labor, with the result that the contractor's profit was saved in every instance. A contract would be let out to build in one street a sewer which was of the same nature as was being built upon another street by day's labor; and this proved the case clearly.

The keeping of these accounts has had the following result: Each foreman, when his particular work is finished, knows just how much it has cost, and, if this cost exceeds the cost of similar work under another foreman, he knows at once that he is not up to the standard, and a continuance would mean the loss of his job; but, aside from this fear, this system makes each foreman an enthusiastic worker for a reputation, and results in a financial benefit to his employer. If the cost of each superintendent's work is not made known to him, what means has he of knowing whether his work is or is not successful? And unless the engineer keeps a careful record of the cost, he cannot give the foremen such information, and the burden finally falls upon him.

Mr. Smith. OBERLIN SMITH, M. Am. Soc. C. E.—It may be of interest to change the point of view to indoor work, and speak of manufacturing cost in factories; the speaker's experience having been more particularly in that line. There is a very common belief that manufacturers make enormous profits; that people who run machine-shops, and make goods that outdoor people have to handle afterward, are getting fortunes—not small fortunes, but big ones. It is a common idea, when people send to a machine-shop to have repairs done or machinery built, that if the charge is made by the hour, at say 50 cents or more, there is a profit of about 100 per cent. The ordinary public, and perhaps even some engineers who do not have to do the work, are apt to say: "Behold! Their men are paid 25 cents an hour and they charge you 50 cents, a very nice thing in profits." But they ignore the little items of fixed charges, about which much is not generally known by outsiders. These, however, as a matter of fact, sum up to a very large amount. If we consider a medium-sized machine-shop, employing say one hundred or two hundred men, where (there being a good many boys) the wages average, perhaps, only 20 cents an hour, the expense-rate, or hour's burden, that has to be put on each and every hour worked by producers, is usually about the same amount. That is to say, the sum paid for wages upon a job must have about 100% added, to cover ordinary running costs. One modern way, therefore, of finding costs in such establishments is to take the material for any given job, at its actual cost, delivered at the factory, including freight, waste, etc., plus the wages actually paid, plus the "burden" on each hour worked by those men in the factory who really produce something. There are, of course, a good many non-producers whose wages go to expense account. This burden is very often put at 20 cents an hour. It may run as low as 15 and as high as 25, but 20 seems to be

a somewhat safe average for the ordinary medium-sized machine-shop in the United States. A burden rate is ascertained by taking the total expense of running the business, outside of material bought and wages paid. This usually consists of: interest on capital, or, perhaps, rent; insurance; taxes; commercial expenses of selling (the most variable item in the whole account), including advertising, traveling, bad debts, part of office expenses and salaries of officers and clerks, together with many sundries; repairs to machinery and buildings (a very large item); fuel, oil, light and other consumables; wages of non-producers about a factory, as engine-men, janitors, etc., etc. Mr. Smith.

These items, added up at the end of the year, divided by the total number of hours' work made by the producing class, will give the hourly burden. Of course, it usually comes out in fractional figures, but an approximate rate, in round numbers, is good enough – if set high enough. The burden for each year is based upon the previous one, and hence cannot be really accurate. It will be seen, therefore, that a machine-shop man who uses medium-sized tools, may very likely find his work costing him 40 cents an hour, 20 cents for wages and 20 cents for expense. In some shops, however, the wages will average 25 or 30 cents, and so the cost may be 45 or 50 cents. Hence, with a not unusual charge of 50 cents an hour, there is a question whether there is a total net profit of as much as 10 per cent.

In using very large machine-tools the average price per hour is put higher because of the greater amount of room these machines take up, and their greater interest and repair account, to say nothing of their likelihood of standing idle more of their time. The machine-shop man, therefore, sometimes perpetrates a charge of $1.00 an hour, or more.

Hence those of the outside public who use machines should understand that the profits on making them are not abnormal. They are moderate, and it takes a good deal of time and care to find out what they are, and much good management to keep things going so that the profit comes out on the right side and does not prove to be a minus quantity.

Charles Worthington, M. Am. Soc. C. E.—One of the most important acts an engineer has to perform is the preparation of accurate estimates of preliminary cost, in connection with any work he may be called upon to design. His clients, of course, want to know at the outset what the work is going to cost, and, possibly, how certain variations in design will affect the total cost. The engineer, therefore, should be prepared to make these estimates on short notice. To determine the cost it is necessary to have, first, an accurate estimate of the quantities of materials required in the work, which is a matter of calculation; and then accurate unit costs which will apply to the items making up that estimate. To determine these unit costs it is necessary to look to the past experience, either of the engineer himself Mr. Worthington.

Mr. Worthington. or of others. In some lines of work, as in bridge manufacturing, the unit costs have become quite generally known, but in many other lines they have not; and engineers who publish the unit costs of work actually constructed, with specific descriptions of the local conditions affecting the same, do a great service to the profession at large. Many of the statements of cost which appear in the engineering periodicals from time to time have little value, for the reason that the unit costs are not given, and local conditions are not clearly specified.

Mr. Crowell. FOSTER CROWELL, M. Am. Soc. C. E.—There is one consideration of this subject which is important: A great many engineers are deterred from an attempt to ascertain the unit prices of the work in their charge, because they think it is a very difficult operation to collect and arrange the data. That is true if the desire is to have an absolutely accurate account which will balance, but, for the purpose in view, it is not necessary that that degree of accuracy should be attained. If such accuracy were to be attempted it would necessitate a duplicate set of accounts, adding much to the cost, and occasioning very great difficulty in getting the accurate data; but it is quite possible to reach a sufficiently close approximation to the cost, and to the average price.

Mr. Oberlin Smith has spoken of the figures of cost which he had obtained in a given case and was quite content to express it in terms of an easy average. He did not think it necessary to go down to a decimal of a cent or even to a cent. Now, if engineers in charge of work educate and instruct their staff to collect certain items of information as the work proceeds, it will be found quite easy at the end of the work to approximate the cost of any one of the items, or all the items, with sufficient closeness for comparison and for reference in undertaking other work of the same kind.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E.—The speaker would answer this question in the affirmative; and, if the affirmative answer to the question relating to Separate and General Contracts is accepted, there is no difficulty whatever in keeping the accounts.

In commenting further upon these two subjects, it may be said that the more experience one has in carrying on work, as suggested, the more will it inure to the benefit of the owners and to the knowledge of the engineer.

Mr. Thompson. SANFORD E. THOMPSON, Assoc. M. Am. Soc. C. E. (by letter).—Too great stress cannot be laid upon the utter worthlessness of any system of cost-keeping records which does not describe in full the materials handled and the conditions under which the work is performed.

The writer calls to mind a large cost-sheet relating to an earth dam with a concrete core-wall and a masonry spillway, which was constructed under contract, for storage-basin purposes, by one of our large cities. The engineers for the city had kept carefully the cost of each division of the work, and the unit costs were calculated. The cost

per cubic yard for hauling the earth was calculated very carefully, but no remarks were made upon the character of the material excavated or the length of the haul. The actual cost per cubic yard of the masonry was recorded, but no distinction was made between face work and rubble work. The same lack of data was found in all the other divisions of the sheet. The unit costs were, in other words, of no use whatever in estimating another job, unless the design and the conditions under which the work was to be constructed were practically identical. Mr. Thompson.

This is cited merely as an illustration of the numerous cases where time is wasted in making up records, which, while mathematically exact, give no results that can be used for the basis of estimates upon new work which is different and yet contains similar features.

In the annual reports of almost all city engineers there are tables of costs of pipe laying, paving, and other municipal work, which would be of considerable value to the profession if such details as depth of cut, character of materials, length of haul, and laborer's rate of wages per day were given.

The experience of the writer has been that to obtain unit costs of real value, either in factory cost-keeping or in engineering construction, careful observations, covering short periods of time but taken very accurately, with all the attendant conditions described fully, are of far greater value for future estimates than figures which cover a long period of time but are not itemized carefully and for which the work is not described accurately. Considerable experience and practice are necessary, in taking either indoor or outdoor records of times of short duration, to select average conditions, and to make sure that all the auxiliary operations of the work are included, and that sufficient allowances are made for the unavoidable delays occurring throughout the day.

W. W. Cummings, M. Am. Soc. C. E. (by letter).—The subject of "Unit Costs of Work" is worthy of much thought. The writer heartily agrees with Mr. Whinery that the knowledge of such units, as the work progresses, often leads to discoveries of want of proper management, or lack of previous knowledge of the difficulties to be met, which results in prompt discipline in the one case or more economical methods in the other, and which, without such a system, would not have been appreciated until it was too late to remedy the evils. Mr. Cummings.

In harmony with Mr. Churchill's observations, the writer has known of ordinary second-rate foremen who have been stimulated into producing results of a high order by the daily comparison of the costs of their work.

It seems as though the discussion of advisability and possibility might be extended into a synopsis of the methods of securing these units and the principles on which such methods are based.

Mr. Cummings. In establishing a system of reports, as has been well said, the general outlines must depend on the character of the work in hand, but the underlying principles are the same in all cases. No one but a person who has struggled through the establishment of such a method can appreciate the amount of labor necessary to avoid the complication caused by too much detail, or the ambiguity of too little.

Simplicity is the first requisite, both on account of the uneducated men on whom the engineer must depend to secure the first entries, and for the purpose of making the clerical labor a minimum. A great mistake is to aim at such accuracy in obtaining the first entries that the notes become complicated and confused. As Mr. Crowell has said, extreme accuracy is not necessary, nor is it desirable. With a little coaching, the foreman will use his judgment in charging the odd fractions of labor and material, so that the weekly average will be more nearly correct than if a large mass of detailed data were obtained.

One principle that is important is to have the divisions and subdivisions of the reports on the same general plan, so that, in the gradual combination of the diversified reports, it is only necessary to add the similar items from the different parts of the work and for the different days, weeks, months, etc., until the work is completed.

Another principle is to combine the statistics by "daily" totals, and "to date" totals, "grand totals," etc., so that the subdivision of costs is obtained at once and can be read at a glance.

In devising a system to apply these principles the engineer takes the items from which he made his estimate, and has a daily report card from each subdivision or foreman. On these cards the labor and tools are classified, and there are seven columns headed: Number of Men, Previous Amounts, Hours, Rate, Daily, To Date, and Remarks.

Each foreman enters the hours of labor and the material for his section for the day. His section comprises, if possible, a certain condition of work. The time-keeper fills out the "Previous," "Daily" and "To Date" columns, and, under "Remarks," gives the qualifying conditions.

The cards are sent to the office, together with the engineer's reports of the "Daily" and "To date" work done in the corresponding classes. The book-keeper enters the totals, from the foremen working under similar conditions, on a page ruled into sections of three columns each and having a particular class of the work for the heading of each section. The three columns are headed "Amount," "Rate" and "Total," respectively. On one side of the page is a column for dates and a wide column for subdivided headings, while on the other side of the page is a wide column for remarks. This page is headed like the different foremen's cards, and can be used for daily, weekly, monthly or yearly reports by simply transcribing the totals.

Mr. Cummings.

The writer has used such a system for a number of years, and has been agreeably surprised at the ease with which the foremen take up the idea, and the small clerical force necessary to keep the records up to date. He recalls one time when 1 500 men were employed on a line extending over 12 miles, and the entire force consisted of three division engineers with one and two assistants each, three time-keepers, one paymaster and two book-keepers. At that time the daily records were complete for inspection by 12 o'clock the following day.

Mr. Whinery.

S. Whinery, M. Am. Soc. C. E. (by letter).—The discussion of this topic has been interesting and valuable, but not as full as the importance of the subject merits.

If some member of the Society, who has given the subject attention and has had considerable experience in devising and applying cost-keeping systems on engineering works, would favor the Society with a paper upon the subject, going into details and illustrations, he would render a substantial service to the Profession.

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TRANSACTIONS.

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RELATIVE PERMANENCE OF STEEL AND MASONRY CONSTRUCTION.

An Informal Discussion at the Annual Convention, May 21st, 1902.

SUBJECT FOR DISCUSSION.

"Is steel susceptible of being made as permanent a building material as masonry?"

By MESSRS. CHARLES G. DARRACH, EUGENE W. STERN, GEORGE F. SWAIN, CHARLES C. WENTWORTH, OBERLIN SMITH, WILLIAM R. WEBSTER, JAMES OWEN, E. T. D. MYERS, JR., W. HILDENBRAND, H. S. HAINES, A. L. JOHNSON, F. LYNWOOD GARRISON, J. F. O'ROURKE AND CHARLES G. DARRACH.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E.—The speaker approaches this subject with some trepidation, as it is worthy of higher talent than he possesses.

To deal with the subject properly, there should be not only discussion, but contention and consultation, so that it may not be recorded among the dead by the epitaph which heads our publications: "This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications."

The speaker would suggest that the subject be amended so as to read, "Is Metallic Construction, or the Combination of Metal and Cement Concrete, Susceptible of Being Made as Permanent a Building Material as Masonry?"

Many years ago, the late Julius W. Adams, then President of the Society, suggested to the speaker that it was pre-eminently the business of the engineer to adapt the *motif* of his construction to the genius of the locality in which his field of operations lay. This is, in fact, the chief business of the engineer, and is using the best and cheapest means for the end to be attained. Mr. Darrach.

It must not be forgotten, however, in making comparisons, that the best materials may be manipulated so as to produce inferior results, and the proper use of materials in composite engineering construction is as absolutely necessary as in the domestic arts, which please our palates, for it is well known that the cook can either spoil the pie or make the pudding.

Before determining upon a method of construction, there should be considered, not only the first cost, but also the expense of maintenance. The ease of inspection during erection should also be considered, as well as the adaptability of the material to attain the most economical results.

A knowledge of the relative strength of the various materials, and also of the causes for deterioration and decay should be attained. With this knowledge the problems will reduce themselves to some degree of simplicity. This knowledge can then be supplemented by observation and experiment, so that there need be no reason to fear results. Knowing the disease, the remedy therefor can be obtained.

The idea that masonry is the *sine qua non* of all permanent construction seems to the speaker to be without foundation. Permanence of masonry construction depends, not only upon proper design, but also upon perfect workmanship. All who have had any experience in construction know well that the most trying and difficult task of the engineer is to have masonry properly erected. In masonry construction, dependence must be placed upon the physical value of the stone, sand and cement. The stone, being a natural product, may or may not have that degree of permanence expected by the engineer, and, to meet this contingency, a large factor of safety is generally introduced in the calculations for masonry construction. Also, in many cases, where masonry is used, it is impossible to design the structure so that the economical quantities of material to meet necessary requirements can be used.

As has been intimated, masonry, as well as metallic construction, may be subject to deterioration and decay.

In using metallic construction, it is possible to calculate the stress upon the structure with a far greater degree of accuracy than can be applied to a masonry structure; consequently, the factor of ignorance can be reduced and the quantity of material used be nearer to that which is theoretically required. Inspection during con-

Mr. Darrach. struction and erection, and after completion, is more convenient, and, knowing the causes for deterioration, the necessary remedies and protection may be readily applied.

The use of concrete, or artificial stone, appeals to the speaker with much force. By its use artificial stone of equal value throughout the entire content can be obtained, and that value can be determined in direct accordance with the material of which it is made; so that the composition of the artificial stone can be adapted directly to the use for which it is intended. The cost of construction for equal values in artificial stone is generally much less than if natural stone were used.

Artificial stone, or concrete, forms a protection against the usual deterioration of metallic construction by atmospheric influence.

The logical conclusion seems to be that the best method of construction would be composite, using artificial stone and metal. Artificial stone, or concrete, not only forms a protection for the metal, but adds its value in resisting stress, and, except in subterranean and subaqueous structures the deterioration of metal encased in cement concrete of proper composition may be disregarded.

There are many examples of structural metallic work encased in concrete where no deterioration is shown, even from ancient times. The modern system of iron construction encased in concrete or brick masonry, used in high office buildings, shows that, when properly constructed, there is no observable deterioration in the metallic work. The Drexel Building, at Fifth and Chestnut Streets, Philadelphia, was constructed in the years 1886–87. The speaker has removed portions of the ironwork in that building, and could see no deterioration whatever.

In the year 1891, the speaker made a cement floor and walk over a pipe tunnel, the upper surface of which was exposed to all atmospheric influences. This floor was of cement concrete, about 4 ins. in thickness, and had a span of about 5 ft. It was reinforced by bands of ordinary chicken-wire fencing, 16 ins. in width, laid across the tunnel and spaced 16 ins. apart. A bed of mortar, composed of cement and sand, about 1 in. thick, was laid upon a horizontal plank center, and brought up flush with the top of the side-walls of the tunnel. Over this mortar the chicken-wire was laid, extending over the side-walls. The cement concrete floor, 3 ins. in depth, was laid upon this chicken-wire, and finished, the walk being about 7 ft. wide. Although this walk has been in use for eleven years, no deterioration is apparent, nor has there ever been any leakage, even during the most severe storms.

In relation to this, attention is called to what the speaker considers faulty construction. The concrete floors of most office buildings are of cement, ashes or cinders. The resultant concrete is not

impervious. The large quantity of water used in construction is retained within the exterior skins of these floors for a long period, and, no doubt, is a detriment to the metallic construction of the floors. Mr. Darrach.

In the year 1884, the speaker constructed a water-works well at Redbank, N. J. This well was some 15 ft. in interior diameter, and was sunk from the surface of the ground to a depth of about 60 ft. The lower half, or 30 ft., was below the ocean level, and the entire well, to within 5 ft. of the surface of the ground, was under the ground-water level. The water arose in the well to within 8 ft. of the surface of the ground, so that the well casing, 20 ins. in thickness, constructed of hard brick laid in Portland cement mortar, was continually in contact with water on both sides, and at times, even during construction, was under a pressure from the outside of more than 40 ft.

For reinforcing the brick shell during sinking, ⅞-in. iron rods were extended from the cutting curb upward through the masonry; and, at intervals of about 6 ft., large wrought-iron washers were placed over the rods and extended around the entire circumference of the well casing. This construction was adopted to reinforce the green masonry of the casing during the operation of sinking. It would be interesting to examine this ironwork for the purposes of discussing this subject.

In the speaker's opinion, the hue and cry raised in relation to a properly constructed iron skeleton, in modern high buildings, is without foundation.

In buildings where the ironwork is concealed within fire-proofing or concrete, there is little reason to suppose that any deterioration takes place, and the speaker will venture to state that, if the temperature and the humidity of the atmosphere immediately adjacent to the metallic structural work were observed and recorded, there would be found but little, if any, difference throughout the year.

Greater care, and concrete richer in cement, should be used for subterranean and subaqueous composite construction, and, in some cases, it would be advisable to use an asphaltic concrete, so as to absolutely prevent the possibility of any moisture coming in contact with the metal.

The numerous cases of electrolysis in subterranean metallic pipes suggest a criticism which has not as yet been presented to the speaker. Is it not possible that, in subterranean and subaqueous work, metal, even when constructed in cement concrete, may be subject to electrolysis similar to that experienced in metallic water and gas pipes? The speaker is of the opinion that all such subaqueous and subterranean constructions should be laid, as heretofore indicated, in an asphaltic concrete, and efforts should be made to force our legislators to pass

Mr. Darrach. laws preventing the use of the ground for an omnibus commercial electrical conductor.

We can also substitute for cast and wrought-iron pipes a cheaper and more permanent conduit, constructed under the composite system. The capacity of metallic water pipes is materially diminished by the formation of nodules on their interior surfaces, and, with some waters, the life of iron pipes is merely a question of a very few years.

Composite pipes, constructed of concrete reinforced by metal, can be constructed so as to have the necessary strength without the loss of conductivity due to the metal nodules, and it is not necessary that the concrete of which they are composed should be of such richness as to absolutely prevent at first the transmission of moisture, except when the liquid conducted is free from suspended matter. Conduits carrying either turbid water or sewage soon fill up the pores in the concrete, and make it impervious.

In 1894, the speaker constructed a septic tank for the Insane Asylum at Wernersville, Pa. The diameter of the tank was 25 ft., and the depth of water was 14 ft. The well was constructed of hard brick, laid in Portland cement mortar, and had a maximum thickness of 21 ins. There was an inside coating of plaster, ½ in. thick, made of equal parts of Portland cement and sharp sand. Tests were made, through a period of ten weeks, at the end of which time the masonry was practically impervious.

There is a paucity of experimental knowledge as to the strength of composite construction; the field is wide, of great importance, and deserves the attention and study of the members of our profession.

Before closing, the speaker would suggest that further experiment be instituted as to the possibility of making a metallic weld, to avoid as much as possible the use of bolts and rivets. He is aware that the electric weld does not give a resultant value of 100%, but he is of the opinion that the Society would be gratified with a knowledge of the best results thus far obtained.

Mr. Stern. EUGENE W. STERN, M. Am. Soc. C. E.—Discussing this question from the standpoint of the protection of iron and steel construction in buildings, the experience of the speaker is that iron does rust in buildings. That is, it rusts unless properly protected. Therefore, it is a very important question to determine what kind of protection it needs to prevent this, under different conditions of exposure.

In the interior of buildings, where there is no chance of moisture or acids attacking the iron, and where it is usually surrounded by fireproofing materials, the danger from rust is very slight. The usual coat of paint appears to give all the protection necessary.

The speaker has seen ironwork taken from the interior of buildings, in one case after thirty years, in which the shop marks of white paint and the brown coat of red oxide paint were in as perfect condition as if done but recently.

In the exterior walls of a building, however, and in the roof and cellar construction, it is an entirely different matter. A driving rain will go through the outer coating of brickwork; roofs will somehow or other leak, and cellars rarely are absolutely dry. The speaker has invariably seen evidences of rust on the ironwork taken from the above-mentioned positions in comparatively recent buildings. The paint seemed to have disappeared entirely in many cases where exterior brickwork was laid up against the iron. Where the mortar was in immediate contact with the iron, there was no rust; where the brick was directly in contact, there was considerable rust. Mr Stern.

In a building in New York City, built in 1869, and now being taken down, cast-iron girders supporting the brick arches of the sidewalk construction were rusted about ⅛ in. where the brick was immediately in contact with the iron, but where the mortar was directly in contact the iron was perfectly clean and black, looking as if taken from the sand only yesterday. The evidence that paint protects ironwork thus exposed, for any length of time, is not forthcoming. The paint seems to disappear entirely in a few years. Nor, in the speaker's opinion, does it help matters to prepare the surface of the ironwork, prior to painting, in the most painstaking and thorough manner, even using the sand-blast to remove the mill scale. Some of the paint is rubbed off the ironwork during erection, notwithstanding that the very greatest care is used. It is an inseparable condition, from the nature of the work, that this happens. There is then a starting point for rust, and this will follow more quickly on a sand-blasted surface than on one having its mill scale.

The speaker is strongly opposed to the use of the sand-blast on ironwork for buildings. It does not help to protect iron permanently, and the expense, therefore, is not justified. For this reason the following method of protection is recommended: All the columns and girders in the exterior walls, the roof, the cellar, and wherever exposed to moisture or acids, should be surrounded with Portland cement mortar, and the bore of the columns in such locations should also be filled solid with this mortar.

Broken stone in the mixture is not recommended, because the voids may not always be filled; and cinders are strongly objected to, because there is good evidence to show their very harmful effect.

A building having its frame thus protected should remain structurally sound, judging from evidence, for many years, perhaps longer than one made entirely of masonry, unless of the best quality.

George F. Swain, M. Am. Soc. C. E.—People in general have come to consider iron or steel as rather perishable materials when compared with stone. Many persons form this idea by considering the old stone structures of the Greeks and Romans, notwithstanding the fact, referred to by one speaker, that there are a great many stone Mr. Swain.

Mr. Swain. structures which have proved very perishable. Many instances can be cited of bridge abutments built of sandstone or limestone which have practically gone to pieces within thirty or forty years. In judging of the ancient stone structures of Greece or Rome, we must also remember the difference in climate between those countries and Northern New England. It may not be out of place to point out also some differences between the ways in which steel and stone structures act. A stone structure is a comparatively simple one; it acts by its own weight; it acts generally in simple compression; its durability is a question of stability rather than of strength, and the loads may generally be increased greatly without endangering it.

A steel structure is very different. It is a complex structure, involving complicated stresses and strains; its strength is a controlling factor in its durability, and any increase of load reduces correspondingly its margin of safety. Engineers who have had much to do with questions of renewing bridges will agree with the speaker that cases in which bridges have had to be renewed on account of the actual wearing out of the material are comparatively rare. Of course, there are cases where corrosion is active, especially in bridges over railroads, but, leaving out such cases, the greater number of renewals which have taken place are due to other causes. One of these is defective design. Engineers are learning more and more every day in regard to the details of steel structures, and they are experimenting and studying the various connections and details. The result is that we find that structures built twenty or thirty years ago, and supposed to have an ample factor of safety, really had a much smaller factor. Moreover, as everyone knows, the loads to which steel structures are exposed have increased enormously. These are the two elements which have caused the principal renewals of bridge structures. These elements will continue to be present, and, as long as they are present, we cannot hope to have steel structures in general as durable as the best stone structures would be. In the case of a steel-frame building, the loads, presumably, are not to change very much, and with good design the actual durability of the material should govern the life of the structure. As to what that durability will be seems very uncertain. All the engineer can do is to take every precaution he can to insure the permanency of his steel. The speaker agrees with Mr. Stern that the steel should be completely enclosed in concrete, but, even under these conditions, he does not think the steel will last as long as the best masonry.

Mr. Wentworth. CHARLES C. WENTWORTH, M. Am. Soc. C. E.—The answer so far given to this question is that steel, when embedded in concrete to a sufficient depth, is permanent. A structure in which the steel is thus hidden becomes at times as essentially one built of concrete as of steel, and may be even more so. The fact that, in most cases, it is

impracticable to adopt a composite construction, rather than one all steel or all concrete, renders it still necessary to seek a more complete answer to the question; especially if it be considered, as it must be, that bridges are made of "building material." Mr. Wentworth.

The answer given, for instance, is inapplicable to the East River bridges, built or building in New York City, and to the greater part, by weight, of ordinary railway bridges. It is surely inapplicable to more structural steel work, by weight, than it is applicable to all kinds.

The protection afforded by concrete is afforded by paint, but in a less degree. The objection to paint as an answer to the question lies in the fact that much structural work is inaccessible after erection, and that, in all likelihood, paint will not be applied properly or with sufficient frequency even where the work can be reached by the painter.

A more complete answer to the question appears to lie in the electroplating of structural steel with copper or aluminum. This has already been done, to a small extent; but the subject does not appear to have received the attention it deserves.

The speaker sees no reason why such plating need be of excessive cost. Aluminum is an exceedingly abundant element, and is becoming cheaper; copper may retain its value. Metallic aluminum may not be needed in the process, but such aluminum salt as will serve when in solution as the source of the metallic aluminum finally composing the plating.

Of course it would be impracticable to plate steel as it comes from the rolls, as rivet heads and sheared edges would need subsequent plating; but each structural piece, ready for shipment and erection in the structure, could be plated, and the details of field connections arranged with a view to their permanency on the same lines.

The speaker does not wish to appear as objecting to the use of concrete structures stiffened by steel, with the incidental protection afforded to the steel by the concrete. Such structures are entirely proper and appropriate in very many cases; but, in conclusion, he wishes to say that the plating of structural steel, as a more general answer to the question, is well worthy of the attention of the members of this Society.

Oberlin Smith, M. Am. Soc. C. E.—The speaker is not experienced in putting up big steel buildings, but it seems to him that a certain precaution should be taken, which, perhaps, usually is not, and which will be mentioned later. Some of these buildings have been up a good many years and seem to be very strong, but after this generation is gone—fifty or one hundred years hence—if there is no way to examine all that hidden metal inside the masonry, how are people Mr. Smith.

Mr. Smith. going to feel about the stability of such buildings? When will the time arrive for them to come down of themselves? There should be, therefore, some systematic method of examination.

Of course, after many years, the owners may tear out some of the masonry and examine the steel, but it would be a small matter now to provide inspection holes, properly covered, and to differentiate and classify the various conditions of possible deterioration to which the metal may be subjected.

Certain parts of the building which are most likely to rust, and where such rusting is most likely to cause accidents and damage, would be in one class, while other parts which are not so likely to be affected would be in another, and so forth. In every building, in several proper locations, let there be certain places left where the metal can very easily be exposed, and again covered up—of course in a fire-proof manner. This would be done instead of merely tearing away a building at random, with perhaps considerable damage and with consequent expensive repairs.

To find out how the steel is really getting along, there should be certain times for observation, with definite intervals between. These intervals might be five, or ten, or twenty-five, years—as experience should indicate. The most risky "classes" should be examined the oftenest. In general, there should be certain definite methods of inspection, and definite places for inspection, and definite times for inspection of this buried and perhaps much-suffering metal, not only that such may be done with less damage to the building, but that it may be done thinkingly and systematically—instead of waiting for some Chicago Post Office to become obsolete that we may see whether it continued safe until the end. If not done in such a manner all may be neglected, and intervals which are uncertain may become of infinite length.

Mr. Webster. William R. Webster, M. Am. Soc. C. E.—The examination of many old bridges and other structures has shown that the white-lead marking letters of the mill are intact, and that the metal is better protected at these places than at others. That is, the plates and shapes are generally marked while hot, and before the material has had a chance to rust; but the full importance of applying the protective coating while the metal is hot has not been appreciated.

The ordinary oiling at the mills has not been satisfactory, as the material is generally made sticky and difficult to handle in the shops. In relation to this, attention is called to a railroad spike dipped in oil, while slightly heated (in accordance with the specifications for an export order). Under this treatment the oil quickly dried and formed a good, hard protective coating. This process might be applied to ordinary structural shapes and plates with great advantage; the material being cleaned with brushes and an air blast, and the oil applied

while the material is still hot, the coating in no case being heavy enough to interfere with a thorough inspection for surface defects. The shop painting, of course, should be applied in the usual manner. Mr. Webster.

Electric welds have been referred to as being practicable for welding beams to beams, or beams to columns, in buildings. This would be a step in the wrong direction, as reliable welds of this character cannot be made. The size of the grain is raised during the heating, and there is no chance of putting the required work on it to break up the grain and produce the fine silky structure. That is, electric welds would be fatal to any work that is subject to shock.

James Owen, M. Am. Soc. C. E.—It might be interesting to know that, in endeavoring to find some records of the strength of concrete beams, the speaker found, in the library of the Society, the report of some experiments made by Mr. Kirkaldy in 1854. Mr. Kirkaldy at that time had made an elaborate investigation of the strength of the admixture of concrete with iron rods and iron wire. Results showed at that time that the strength of the combination of the two materials, iron and concrete, was increased by more than two and one-half times that of the simple concrete itself. This is quite in accord with the present experiments in that line, and it is interesting to note that while these investigations were made so many years ago, it is only of late that the profession has appreciated their importance. Mr. Owen.

The speaker had a peculiar experience in relation to the strength of an ordinary bridge built with iron beams and brick arches and covered with concrete. The span was 15 ft. The bridge had been built some eight or ten years when the local engineer decided to change the grade of the street, and raised it about 6 ft. This alteration imposed on the bridge itself an extra load of 900 lbs. per square foot. That, with a previous allowance of 200 lbs., made a total of 1 100 lbs. The bridge was originally designed for a live and dead load of 300 lbs. and a factor of safety of four. A critical examination of the bridge, after the extra load had been on about six months, showed no rupture of any kind and no apparent deflection. The bridge is still in use.

It may be presumed, therefore, that the combined use of steel and concrete is susceptible of a great deal of further development, and that its ultimate strength is apparently unknown.

E. T. D. Myers, Jr., M. Am. Soc. C. E.—During the summer of 1901, a rifle barrel was pumped up through the 20-in. pipe connected with the centrifugal pump which was used in dredging out the historic dock immediately in front of Libby Prison. It arrived at the usual joint, just in front of the pump, which was intended to stop obstructions too large to pass through the pump. The pump was stopped and the rifle barrel taken out. It was found that the blue enamel on the barrel was intact on at least 90% of its surface, and the sight could be raised and lowered without the slightest difficulty. The date Mr. Myers.

Mr. Myers. on the barrel was 1856. It was probably thrown in the water in 1865.

The water there is fresh, and is entirely free from lime. It is what is known as a "kind" water for boilers. The speaker does not know whether metal would last that way in other waters, but he has pumped up and dug up shells and rifles which were not rusted.

The depth of water in the dock was originally 16 ft., and the depth of the silt was about 4 ft. The pump was working at a depth of 22 ft. It is impossible to state the depth at which the rifle lay, but it probably came from the 16-ft. level. The bottom is composed of sand and gravel, and the silt lying upon it came from the James River, which runs through a red-clay country. The rifle barrel, therefore, had about 16 ft. of mud and fresh water above it.

The speaker does not feel competent to discuss this topic, but he is deeply interested in it, and hopes that the result of the discussion will be the addition of much valuable information.

Mr. Hildenbrand. W. Hildenbrand, M. Am. Soc. C. E.—With reference to the old Roman viaducts and aqueducts mentioned in this discussion, which are frequently quoted as evidence of the longevity of stonework and its superiority over iron structures, the speaker wishes to draw attention to the fact that in other Roman structures may be found equal evidence of the durability of iron. Remnants of bridges over the Rhine, built by Julius Cæsar, have been dug up during the past fifty years, while the work of regulating the bed and current of the Rhine has been going on. In Cæsar's "Commentarii de Bello Gallico" one of these bridges is minutely described, corresponding in construction to what would now be called a timber trestle, consisting of heavy oak logs connected with iron bolts, clamps and spikes. Quite a number of these logs were found under the bed of the river, nearly 2 000 years after they had served for carrying the Roman legions over the Rhine in their attempt to subjugate a free nation, and the timbers, as well as all iron bolts and spikes, were in a perfect state of preservation.

This fact corroborates Mr. O'Rourke's assertion that iron submerged in pure water will probably never corrode.

Of course, every object in this world perishes, or, rather, disintegrates and changes into some other form; but, if building materials can be preserved for several thousand years, it may be said that, for all practical purposes, they last forever.

The question whether metal will in the future be considered as permanent and durable a building material as stone cannot be solved at the present time, and it is not likely that any who are here will live to see it solved. The answer to the question cannot be found by theorizing, but depends entirely on experience, which should extend over centuries.

At present we have, so to say, no experience at all about the dura-

bility of iron or steel structures. It is barely sixty years ago that the first iron bridges were built, and it is doubtful whether a single one of that age is in existence to-day. Nearly all the early iron bridges have been removed and replaced by modern structures, not on account of having been decayed or weakened, but because they had outlived their usefulness, and stronger ones were required to accommodate the increased and heavier travel of modern times. Our experience as to the durability of iron and steel structures, therefore, is very limited, and only of recent date. Mr. Hildenbrand.

If a stone building and a steel building were erected side by side, and both structures were left to themselves, without any attempt to protect them against the effects of the atmosphere, there is no doubt that the stone building would stand very much longer than the steel building. In New York City there is an example in the New York and Brooklyn Bridge. The massive stone towers, which are now more than twenty-five years old, have no protection against the weather and have never been repaired. They are to-day as they were left by the hand of the mason twenty-five years ago, and, according to a recent thorough inspection, they are just as good and perfect as they were on the day they were finished. On the other hand, the steel and wire work requires constant attention; there is hardly a time when no men are seen suspended in the meshes of the wire work repairing something or putting on a coat of protective paint.

The speaker has had the opportunity of examining suspension bridges erected more than forty years ago, and while he found the cables, as a whole, in perfect condition, he also discovered one or two places where, through the accumulation of water or through imperfections in the protective paint, the wires were considerably corroded and needed repairs. All of this shows that metal structures require attentive watching and constant renewal of paint or other protective coverings, and that any oversight or inattention is followed by grave consequences.

With our present knowledge, therefore, we may sum up by saying that unprotected masonry, as well as unprotected ironwork, is perishable, but that the former will last much longer than the latter. However, if iron or steel be well protected, it is known that it will not decay in thousands of years, and will be as durable, if not more durable, than any stonework. Will the means for an efficient and permanent protection of steel ever be discovered, or will a new metal be found which has the same strength as steel and which naturally is not subject to oxidation?

H. S. Haines, M. Am. Soc. C. E.—Mr. Hildenbrand has referred to iron bolts in bridges built by Cæsar across the Rhine. In the Colosseum, built about A. D. 80, the courses of heavy travertine masonry, laid with knife-edge joints, without mortar, were connected by iron Mr. Haines.

Mr. Haines. cramps fairly well protected from the weather. Many of these were cut out in the Middle Ages, but those which remain appear to be in good condition. The vault of the Pantheon, 140 ft. in diameter, is of concrete supposed to be strengthened with iron rods. This vault, built about A. D. 125, is, to all appearances, in excellent condition. From these instances, it would seem that iron, used structurally in connection with stone or concrete masonry, may remain efficient for nearly two thousand years.

Iron is not readily oxidizable in pure water, for the oxygen therein chemically combined is only separable at an excessively high temperature. It is the atmospheric oxygen ordinarily present in water that is the oxidizing agent. Water absorbs carbonic acid gas to an extent equal to its volume, and the water thus acidulated acts powerfully upon iron or steel. It follows that in large cities where the atmosphere is heavily charged with the products of combustion, the falling rain would be correspondingly charged with carbonic acid gas, and, therefore, that the effect of oxidization upon steel construction should be proportionately greater from exposure under such conditions.

Mr. Johnson. A. L. Johnson, M. Am. Soc. C. E.—In reference to a statement concerning the corrosion of the steelwork in the large buildings in Chicago, C. T. Purdy, M. Am. Soc. C. E., was one of the members of a Commission to examine and report on the condition of those buildings. The speaker having talked with Mr. Purdy recently concerning this matter, it may be appropriate to give some of the results as described by Mr. Purdy. His report has undoubtedly been printed ere this, and probably, therefore, there will be no objection to the promulgation of the information at this time. The Commission examined quite a number of the buildings in Chicago and found numerous cases of corrosion in the columns; but it was not developed that the corrosion was confined to the outside columns, or even that it was greater in them than in the interior columns. The beams in the foundations, Mr. Purdy said, were found to be in uniformly good condition. The Commission's recommendation would be that in future all the steelwork for these buildings should be entirely surrounded by a Portland cement or lime mortar, this covering being filled in solid behind the fire-proofing.

In St. Louis a very well-constructed building, erected about twenty years ago, is now being dismantled. The construction consisted of solid external walls, cast-iron columns throughout the interior, and steel beams painted with red oxide of lead paint. The building was eight stories high, and of fire-proof construction, the floor arches being mainly of brick, though there were also some concrete arches. The steel throughout is in a thoroughly well-preserved condition, it having been in all cases entirely embedded in Portland cement mortar or concrete covering. The paint also has been thoroughly preserved,

and in the speaker's estimation an embedment in Portland cement mortar is the only means of preserving that preservative. Mr. Johnson.

Professor Spencer Newberry, Manager of the Sandusky Portland Cement Company, recently gave a lecture* in Chicago, which the speaker regards as one of the most valuable articles, with regard to steel-concrete construction, that has appeared in print for some time, covering, as it does, all sides of the question, and some sides in an entirely new manner; such, for example, as the theoretical considerations involved in the preservation of iron by Portland cement covering. In this article he shows that the cement, theoretically, is not simply a neutral or non-injurious agent, but is actively engaged in preventing the formation of rust.

The speaker's company has samples of steel embedded in cinder concrete, in the form of broken pieces of a test span, made in the fall of 1898, and tested in February, 1899, these pieces having lain on the ground uncovered, subject to the action of the elements, for more than three years. On numerous occasions these pieces have been broken open for the purpose of observing the condition of the metal, the most recent case having been before the Engineers' Club of St. Louis, last January, in connection with a talk given by the speaker; and in all cases, without any exception, the metal has been found to be as bright and clean as the day it came from the rolling mill. The quality of the cinders in this sample was considered very poor, containing a good deal of dirt, and the fine material was screened out. This the speaker regards as desirable, though not absolutely necessary, where cinder concrete is used, inasmuch as, if it is not done, a slight film of rust will be formed on the metal. This film, however, never increases in thickness, but, as it costs little to avoid it, the best practice would call for the screening of the cinders. In the speaker's opinion, the reason for this difference in action is due to the fact that the fine stuff contains finely divided particles of sulphur, which are readily dissolved by water, slightly acidulating the same, and, until the concrete dries out thoroughly, a slight corrosive action is taking place. This is soon neutralized by the influence of the Portland cement, but not before a film of rust, not in itself materially injurious, is formed.

Mr. Darrach has expressed himself as of the opinion that at the present time we are not able to calculate the strength of steel-concrete beams with any degree of accuracy. The speaker, of course, will have to take exception to that statement, he having on numerous occasions endeavored to show how this could be done, and, being of the opinion that the data with regard to the materials used being known, he could arrive at the maximum carrying capacity within about

* Before the Annual Meeting of the Associated Expanded Metal Companies, and reprinted recently in *Engineering News*.

Mr. Johnson. 15%, which is certainly close enough for all practical purposes, in view of the factor of safety used. Of course, the strength and the modulus of elasticity of the concrete and steel must be known, and, as to the former material, these functions are seldom accurately known, chiefly because no special tests are made to determine this for the materials available for that particular work. Their influence on the character of the design is enormous, and, on work of any considerable size or importance, these values should be obtained before the designs are prepared.

As to the relative lasting qualities of the different kinds of engineering structures, the speaker is of the opinion that only a few forms of stone masonry can compare in length of life with a concrete construction reinforced with steel properly distributed through the cross-section. Strange as it may seem to those not conversant with the facts, the maintenance charges for stone masonry, on many railroads, exceed these charges for steel structures in proportion to the relative quantities of each. On railway work it is usually a condition of taking what you can get, rather than what you would like to have. Many forms of sandstone are worthless. Limestone is dissolved in the course of time by atmospheric influences, and there are really only a few kinds of stone that can be considered as everlasting, and these are usually so expensive as to prohibit their use. A very good example of the dissolution of limestone masonry is afforded at the Cabin John Bridge. Underneath the arch may be noticed stalactites forming along the line of the mortar joints between the granite arch stones, from the spring line some distance up toward the crown. The back-filling of the arch consists of a considerable depth of limestone rubble masonry. A considerable quantity of water finds its way down through the haunches of the arch and then through the mortar joints of the arch ring, and the stalactites indicate that the air and water are dissolving this limestone backing. This same action exists, to a greater or less degree, in all limestone structures.

In a large city in Indiana the speaker recently examined some limestone bridge masonry which had been in place for about forty years, and much of it could now be scraped up with a fire shovel.

In the speaker's opinion, plain concrete construction would not be everlasting, on account of the cracks which are almost certain to develop in such structures, these cracks filling with water and freezing in the winter, and gradually getting worse and spawling off as the years go by.

In steel-concrete construction, if the metal is properly distributed and proportioned to the cross-section, these cracks can be absolutely prevented. The speaker's company has built walls 300 ft. long, with steel reinforcement, in which no expansion joints were provided, and in which not a sign of a crack is to be seen;

and would take a contract to build such a wall a mile long, under this guaranty. Mr. Johnson.

To accomplish this result, however, it is necessary to have a subdivision of the metal reinforcement into small areas thoroughly disseminated through the section, just as in successful arch construction it is necessary to have the metal thoroughly disseminated in small areas through the upper and lower portions of the sections. Heavy concentrations of metal at points 2 or 3 ft. apart will not give the peculiar stretching quality to the concrete obtained by the other method, and which is essential to success.

When properly built, this steel-concrete construction will not crack; will not be disintegrated by frost; will not be dissolved by the elements; and, in the speaker's opinion, is the only kind of engineering structure that can be considered permanent, with the exception of one or two kinds of rock masonry, the cost of which in most cases would be prohibitive.

F. Lynwood Garrison, Assoc. M. Am. Soc. C. E.—Since there seems to be some confusion or misunderstanding regarding the corrosive action of atmospheric and other influences upon iron, it might be advisable, in this discussion, to state briefly a few of the most important characteristics of metallic iron that are pertinent to the subject, although, in so doing, the risk of repeating some well-known facts must be incurred. Mr. Garrison.

(1) To begin with, iron is one of the metals most easily oxidized and affected by moisture, consequently it never occurs in the metallic state.*

(2) Iron does not undergo any alteration in pure dry air at ordinary temperatures.

(3) In moist air iron becomes coated with ferric oxyhydrate having approximately the composition $Fe_4O_3(OH_6)$. The rust varies in composition with the conditions under which corrosion takes place.

(4) According to Percy, iron does not rust unless there is an actual deposition of liquid water upon the surface of the metal.†

(5) The presence of certain gases and vapors, even in minute proportions, such as sulphuretted hydrogen (H_2S), hydrogen, chlorine, and acetic acid, accelerates rusting in moist air, though no liquid water may come in contact with the metallic surface. Carbon dioxide (CO_2) and ammonia gas are said to act less energetically in this respect.‡

(6) Iron rust often contains minute quantities of ammonia, due, it is supposed, to the decomposition of the water by the action of the oxide on the metallic iron, the oxygen combining with the iron, and

* Exceptions must be made where the iron is of undoubted meteoric or non-terrestrial origin. In such instances, it is usually alloyed with a large proportion of nickel.

† "Metallurgy of Iron and Steel," p. 27.

‡ Bonsdorff, "Répertoire de Chimie," Vol. 4, p. 171.

Mr. Garrison. part of the hydrogen uniting in the nascent state with the nitrogen of the air.

(7) Pure water deprived of air appears to be absolutely inert, as far as corrosive action is concerned, on contact with iron, even at 100° Cent.*

(8) Rust formed far beneath the water consists of black hydrated magnetic oxide.

(9) The formation of rust takes place in the beginning but slowly; after a thin coating has once been formed, the corrosive process goes on more rapidly.

(10) Aqueous solutions of potash, soda and ammonia, preserve iron from rusting, provided they are not too dilute.

(11) Water containing not more than one-fifth its volume of lime water is said to preserve iron from rusting.†

(12) The contact of iron with more electro-positive substances, such as zinc, retards corrosion; whereas contact with more electro-negative substances, such as tin, lead and copper, accelerates the rusting.

(13) Magnetic and similar oxides of iron, which constitute the basis of iron scale, protect the iron which they coat, but hasten the corrosion of rusted iron, whether such be the adjacent portions of the same piece or in separate pieces which are galvanically connected. This protective action is shown by the comparatively slow rusting of Russian sheet-iron, of "blued" iron, and of castings that retain their original skin.

(14) While contact with zinc and highly zinciferous brasses retards rusting, contact with copper, or brasses rich with copper, hastens it.

(15) According to Martell, the purer qualities of mild steel when used in ship hulls are more likely to be corroded than impure iron. A steel ship requires more care than an iron one. Nickel steel is not so likely to be corroded in salt water as the ordinary and purer grades of steel.‡

This array of more or less well-known facts might be indefinitely elaborated; however, it covers the subject completely in a general way. It seems to be a perfectly sound conclusion that, other things being equal, where likelihood of corrosion is effectually prevented, iron (steel) makes as permanent and durable a building material as masonry. Better, in fact, for, whereas the latter will certainly, in time, disintegrate, the former cannot. Unusual conditions may certainly exist in which the molecular structure of the metal may change; such, however, must be regarded as abnormal and exceptional.

A statement has been previously made in this discussion, to the effect that, inasmuch as iron properly covered with fire-proof material

* Mallet, Rept. British Assoc., 1840, p. 229.

† Gmelin Handbuch, Vol. 5, p. 185.

‡ Journal, Iron and Steel Inst., No. I, 1889, p. 66.

or concrete will not corrode, uncovered iron in the interior of buildings will be immune from corrosion because it is then covered from the weather on all sides. Were walls and roofs absolutely water or moisture proof, in fact, if the buildings were hermetically sealed, top, sides and base, then, and then only, would the interior metal be free from corrosive influences. As a matter of fact, in most cases, the dangers from corrosive deterioration are much exaggerated; the greatest objection to steel buildings is their instability in fires. Mild steel, at a white heat, is about as stiff as cheese, whereas masonry is not affected in the same way at high temperature, and resists collapse to a far greater degree. Mr. Garrison.

The speaker's belief is, that, in practice, a thick covering of cement concrete is the only material that absolutely protects iron from corrosion. Such a compound structure unites the advantages of both component materials, and greatly surpasses in strength and durability an edifice made of either alone.

All are familiar with the remarkable strength and endurance of "wire-glass;" the glass sheet being cast and rolled so that the wire net is completely covered with the glass, imparting to the natural fragility of the latter a certain amount of elasticity and abnormal strength; the glass, in turn, absolutely protecting the wire from corrosion. So long as the wire is covered, such a composite sheet will hold together, unless broken by a force greater than the tensile strength of the wire.

Not being a structural engineer, the speaker may be unfamiliar with certain objections to composite concrete and metal structures; in his ignorance, however, he cannot but think that such compound constructions will be the true and logical line of development in the future.

J. F. O'Rourke, M. Am. Soc. C. E.—Perhaps the principal feature in relation to the durability of steel is the effect of water upon steel in structures. If steel or metal is exposed to the combined action of air and water a condition is brought about that soon produces rust, which is simply the decomposition of the metal. One of the most illuminating examples of the effect of water on metal, with and without air, was brought to the speaker's notice about three years ago. Willson, Adams & Co., lumber dealers on the Harlem River, had put in an artesian well, hoping to get water to take the place of the Croton supply. The water obtained was brackish, but much cooler than the Croton water in summer. It was not fit to drink, but, in order to take advantage of its temperature, they made a U of ⅜-in. pipe, carrying it down about 100 ft. into the well and putting a faucet on its return end. In this way they obtained a pleasant temperature for drinking water all the year round. This pipe was retained in place for, perhaps, ten years. At the end of that time it had rusted off and had to be taken out. The part of this pipe above the water was found to be absolutely rotten. The pipe which had been below the water Mr. O'Rourke.

Mr. O'Rourke. was in perfect condition, and even retained the blue mill scale and tool marks. Upon this part of the pipe, however, there was a whitish coating, like thin whitewash, which could be rubbed off with the fingers. Whether this coating preserved the metal from oxidation, or whether it was the fact that metal does not become oxidized in water alone, are matters for future discussion. The fact is, the metal which had been below water did not show the least sign of decomposition of any kind.

A short distance below the yard of Willson, Adams & Co., the old bridge which crossed the Harlem River at Third Avenue had been supported on pneumatic piles—the speaker thinks they were among the first that were put down in this country. These piles were of cast iron, 6 to 8 ft. in diameter, with interior flanges and bolts to connect them. These bolts were put in in the usual way; and the piles, after being sunk to place, were filled with concrete. The joints were not water-tight; they never are; so everything was immersed, being below the river surface. After 35 or 40 years, when these piles were removed, at the time of building the present structure, the bolts in the flanges were found to be perfectly free from rust. Not only that, they were greasy; the oil on the bolts was there, apparently in the same condition as when put in years before.

The speaker has never known a case where steel or iron corroded when kept away from the influence of the air. He has no hesitation whatever in using steel in connection with structures where there is clean water, if the water remains at a constant level; no more than he has in using wood under similar conditions.

At the present day everybody wants structures built of what might be called high-power material. High power material, in the speaker's opinion, not only possesses high power, but high durability to the same extent, and it only requires care to keep it properly protected against rust. The speaker would use steel or iron under the same conditions that he would use wood.

The durability of pipes is greater when there are inorganic or mineral impurities in the water, than when the water contains organic impurities, as Mr. Darrach has intimated.

The speaker has known of cases, and there are cases on record in the *Transactions* of this Society, where piles which were constantly submerged have rotted, simply because they were exposed to the action of sewage; or, in other words, because they came in contact with organic impurities. Organic impurities act on steel or wood just as a salicylic acid bath would act on masonry. No one will contend that any material is indestructible when in contact with substances which will break it up. Organic impurities will destroy anything. They cause loss of materials and loss of life. Man cannot stand them, nor can iron. Inorganic impurities, or the clean chemical

agents, under certain conditions, will act in a manner that can be foreseen, but with the many organic impurities, few of which are sufficiently known, there is too much uncertainty. When they exist, metal, however protected, should not be risked. Mr. O'Rourke.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E. (by letter).—It is gratifying that upon so important a subject as that under discussion there should be, practically, unanimity of opinion. It would seem that, from the present knowledge of the art, composite construction will receive greater attention than heretofore.

As indicated in the opening remarks, and although in a degree criticised, the uncertainty of calculations, based upon general knowledge, should warn every engineer that failure may result unless special attention is given to, and tests made of, the particular quality of the materials used and the method of manipulation and construction. That the greatest economy can be obtained by composite construction (procuring the tensile values of metal with the compressive and protective values of the artificial stone), both as to first cost and cost of maintenance, seems to be the opinion of those entering into the discussion.

There are two points, however, which seem to have been either overlooked or misapprehended: First, the necessity of protection against electrolysis; and second, that metal is not affected by pure water containing no air. In reference to the latter, it is doubtful whether any engineer ever has to deal with such water. The writer, at least, has not been fortunate enough to have to deal with water containing no air. In his long experience he has not found perfectly pure natural water without air. All natural waters contain air. Clear water, as found in Nature, has a greater erosive tendency than either muddy water or sewage.

Water carrying suspended matter deposits it upon the conduits through which it passes, or upon the surfaces with which it comes in contact, and a part of this suspended matter forms a protective coating on the metal and deters corrosion.

In this discussion several cases have been cited in which metal has laid in silt, under several feet of water, for a long period without corrosion; nor are these results surprising; for, those who have had experience in designing and operating filters know well that with muddy waters, or sewage containing sludge, it takes but a short time and an exceedingly thin film to prevent absolutely the percolation of water through a sand bed.

The Society is to be congratulated upon the very free and intelligent discussion upon this most important subject, and, no doubt, it will be of great benefit to others than those of our own Society.

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TRANSACTIONS.

Paper No. 937.

A PROPOSED NEW TYPE OF MASONRY DAM.*

By George L. Dillman, M. Am. Soc. C. E.

With Discussion by Messrs. H. de B. Parsons, Edward Wegmann, William B. Fuller, George H. Pegram, J. Breuchaud, A. V. Abbott and George L. Dillman.

Strains in masonry dams have been analyzed by many investigators, so that definite requirements must be fulfilled before any proposed plan will receive consideration by the engineering profession. These may be stated briefly as follows :

1.—The dam must not overturn.

2.—The dam must not sustain an intensity of pressure beyond a safe and known limit.

3.—The dam must not slide on its base.

4.—No possible condition must develop tension in any part of the masonry.

It is the purpose of this article to suggest a new type of dam, according to these requirements, as far as they are reasonable, which will contain less masonry, for the same factors of safety, than any of the recognized "standard types."

The theory on which all investigators, from Sazilly to Wegmann, have based their profile sections is that, the stability of one section being proven, a uniform construction to that section would insure the stability of the whole. While this is unquestionably true, it does not

* Presented at the meeting of June 4th, 1902.

follow that uniformity of section is necessary to stability. The writer will strive to show in this article that a more stable and economical dam can be built in the form of a buttressed wall, properly proportioned; or, the proposed type can be considered as a series of short, arched dams between piers shaped so as to be economical of material, and with sufficient factors of safety.

There is a principle in hydraulic construction which is too frequently ignored. It is, to construct one water-tight surface and build the remainder of the structure to support that surface. The attempt to stop leakage that has gotten through such a surface is frequently a cause of failure. If there is leakage or seepage through the upper face of a dam of any of the standard types, the formulas for stability are founded on error. The upper face must be tight, or the computed line of pressure, when the reservoir is full, is wrong. John D. Van Buren, M. Am. Soc. C. E., recognized that seepage under the foundation would also upset the calculations of the analysts of the "standard type" dams, but proposed as a remedy another uniform-section type, very much heavier than any of them.*

While these criticisms also apply to the proposed type, the comparatively thin parts allow this seepage through the upper face to escape before penetration beyond a known point in the masonry, and the seepage under the foundation can escape between the buttresses without exerting a dangerous upward pressure on the base. In either case, the point of leakage can be more easily located.

The writer does not want to be understood as advocating leaky dams; but, as most of the dams in the world are leaky, the possibility of leaks should not be neglected, and their possible effects should be reckoned with.

Infinite variety may result in the application of these principles. One computation will be made here to show the method and the simplicity of the mathematics involved. The solution of no equations above simple ones is necessary. The portions between buttresses will be made parabolic in horizontal section, which will avoid all re-entrant angles, introduce the strongest arch to support the greatest pressure, and, finally, be easier of computation than any other form known to the writer. The crest will be made one-tenth of the height, to agree with the crests of "standard types," approximately. The up-stream

* *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 493.

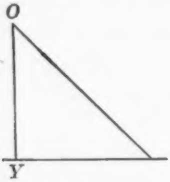
face will be vertical, though no more intricate mathematics is involved by giving it a batter. The line of the arch vertices will be vertical, though a batter to this and the up-stream face would result in additional economy of material. The profile of the buttress faces will be 1 to 1, or 45 degrees. The width of the buttresses at the face will be battered uniformly from the crest, the amount of batter being a matter of computation.

Deduction of Formulas.

Let w = length of one section of the dam, from center to center of buttresses;

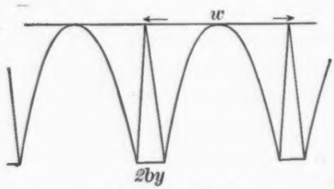
Let y = depth from crest, or height of dam;

Let b = buttress batter, so that $2by$ = width of buttress. Consider the buttresses first (see Figs. 1 and 2).



SECTION THROUGH BUTTRESS.

Fig. 1.



PLAN OF BUTTRESSES.

Fig. 2.

Area of Horizontal Section.

$$A = wy - \frac{2}{3}y(w - 2by) = \frac{1}{3}wy + \frac{4}{3}by^2 = \frac{y}{3}(w + 4by) \ldots (1)$$

Volume from Crest.

$$V = \int_o^y A\,dy = \frac{w}{3}\int y\,dy + \frac{4}{3}b\int y^2\,dy = \frac{y^2}{18}(3w + 8by) \ldots\ldots (2)$$

Center of Gravity of Horizontal Area from OY (see Fig. 1).

Take moments about OY. Let x = the distance of the center of gravity from OY.

$$xA = wy \times \frac{y}{2} - \frac{3}{5}y \times \frac{2}{3}y(w - 2by) = \frac{y^2}{10}(w + 8by)$$

$$x = \frac{3y}{10} \times \frac{w + 8by}{w + 4by} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Center of Gravity of Volume from OY.

$$\text{Distance } V = \int_o^y A\,x\,dy = \int \frac{y}{3}(w + 4by)\frac{3}{10}y \times \frac{w + 8by}{w + 4by}dy$$

$$= \frac{w}{10}\int y^2\,dy + \frac{4b}{5}\int y^3\,dy = \frac{y^3}{30}(w + 6by)$$

$$\text{Distance} = \frac{3y}{5} \times \frac{w + 6\,b\,y}{3\,w + 8\,b\,y} \dots\dots\dots\dots (4)$$

Now add a face to this pier or buttress, of uniform thickness, $\frac{y}{10}$. Its volume is $\frac{w\,y^2}{10}$. Its center of gravity is $\frac{y}{20}$ up stream from $O\ Y$.

Center of Gravity of Face and Buttress, Horizontally from O Y.

Take moments about $O\ Y$.

$$\text{Distance} = \frac{\frac{y^3}{30}(\,w + 6\,b\,y) - \frac{w\,y^3}{200}}{\frac{y^2}{18}(3\,w + 8\,b\,y) + \frac{w\,y^2}{10}} = \frac{3\,y}{160} \times \frac{17\,w + 120\,b\,y}{3\,w + 5\,b\,y} \dots (7)$$

This is down stream from the up-stream toe a distance

$$= \frac{11\,y}{160} \times \frac{9\,w + 40\,b\,y}{3\,w + 5\,b\,y} \dots\dots\dots\dots (8)$$

The weight of masonry is its volume multiplied by its specific gravity. For purposes of comparison, the specific gravity assumed will be the same as that used by Edward Wegmann,* M. Am. Soc. C. E., *i. e.*, $2\frac{1}{3}$.

$$\text{Total weight} = W = \frac{28\,y^2}{135}(3\,w + 5\,b\,y) \dots\dots\dots\dots (9)$$

$$\text{Water pressure} = P = \frac{w\,y^2}{2} \dots\dots\dots\dots (10)$$

[Both (9) and (10) are in terms of cubic feet of water.]

The Resultant of Forces.—When the reservoir is full, the resultant of forces is of the form $z = m\,v + c$, where z and v are the usual co-ordinates (except that z is positive downward), m is a tangent of direction and is equal to $\frac{W}{P}$, and c is a constant. (See Fig. 3.)

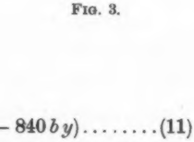


FIG. 3.

When $z = \frac{2}{3}\,y$, $v = \frac{3\,y\,(17\,w + 120\,b\,y)}{160\,(\,3\,w + 5\,b\,y)}$ [see Equation (7)],

thus, $$c = \frac{y}{900\,w}(481\,w - 840\,b\,y),$$

and the equation to the resultant becomes

$$z = \frac{56\,v}{135\,w}(3\,w + 5\,b\,y) + \frac{y}{900\,w}(481\,w - 840\,b\,y) \dots\dots (11)$$

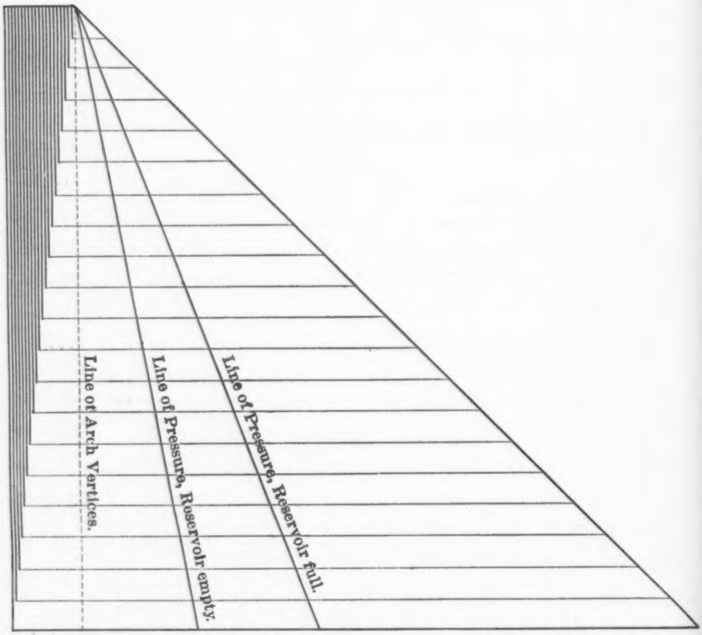
* "The Design and Construction of Dams, Including Masonry, Earth, Rock-fill, and Timber Structures, also the Principal Types of Movable Dams." Fourth edition, 1899.

This intersects the base, $z = y$, at

$$v = \frac{y\,(1257\,w + 2520\,b\,y)}{1120\,(3\,w + 5\,b\,y)} \quad \text{(12)}$$

This is up stream from the down-stream toe a distance

$$= \frac{y\,(2103\,w + 3080\,b\,y)}{1120\,(3\,w + 5\,b\,y)} \quad \text{(13)}$$



SECTIONS THROUGH BUTTRESSES OF DAMS FROM 10 TO 200 FT. IN HEIGHT, SHOWING LINES OF PRESSURE.

FIG. 4.

Intensity of Pressure.—With both dam and foundation rigid, the intensity of pressure at either toe is the average pressure multiplied by the fraction the numerator of which is four times the width of the base minus six times the distance of the center of pressure from the toe, and the denominator of which is the width of the base. In this case, it applies to the up-stream toe, but the factor $\frac{w}{2\,b\,y}$ must be

applied to the lower toe, because the pressure is borne on a length of toe $= 2\,b\,y$, instead of w. The average intensity of pressure is the weight divided by the base, or

$$I_{av.} = \frac{56\,y}{297\,w}\,(3\,w + 5\,b\,y) \ldots\ldots\ldots\ldots\ldots (14)$$

When the reservoir is empty, the intensity of pressure at the upper toe is

$$I_{max.} = \frac{7\,y}{3267\,w}\,(759\,w + 440\,b\,y) \ldots\ldots\ldots\ldots\ldots (15)$$

When the reservoir is full, the intensity of pressure at the lower toe is

$$I_{max.} = \frac{1083\,w + 3080\,b\,y}{6534\,b} \ldots\ldots\ldots\ldots\ldots (16)$$

With these formulas, by substituting any desired value for the maximum intensity of pressure, the ratio $\frac{w}{b}$ can be obtained. w should be taken to make a good proportion with the height of the structure, then b can be calculated. These weights and pressures are all given in terms of cubic feet of water, so that the maximum intensities should be in the same units.

Sliding on the Base.—The angle of the resultant with the vertical is the angle of friction necessary to stability. Call this angle a. Then tan. a is the coefficient of friction necessary to prevent sliding on any horizontal base. Or [see Equations (9), (10) and (11)],

$$f = \tan.\,a = \frac{P}{W} = \frac{1}{m} \ldots\ldots\ldots\ldots\ldots (17)$$

Explanation of Table No. 1.

For reasons given below, $\frac{w}{b}$ will be taken as 1 000 in Table No. 1. Equation (2) gives the volume of a buttress. This, added to the face, $\frac{w\,y^2}{10}$, is the volume of the dam for a length w, so that the volume per foot of length will be

$$\frac{v}{w} = \frac{y^2}{18}\left(3 + 8\,y\,\frac{b}{w}\right) + \frac{y^2}{10} = \frac{y^2}{2\,250}\,(600 + y).$$

Column 2 of Table No. 1 gives this.

Equation (16) gives the intensity of pressure at the lower toe when the reservoir is full. When $\frac{w}{b}$ is 1 000, this becomes, in tons of 2 000 lbs., $\frac{135\,375 + 385\,y}{26\,136}$, which is given in Column 3.

The formula for maximum intensity of pressure when the reservoir is full (taken, so far, to agree with Wegmann's hypothesis), is

only the vertical component of that intensity. The actual pressure is the weight multiplied by sec. a (see Equation 17), and is borne on a base which is the base so far considered multiplied by cos. a. Thus, the intensity of pressure as above computed should be multiplied by the square of sec. a for the actual intensity of pressure. This is given in Column 4.

Equation (15) gives the intensity of pressure at the upper toe when the reservoir is empty. When $\frac{w}{b}$ is 1 000, this becomes, in tons of 2 000 lbs., $\frac{7\, y\, (1\,725 + y)}{237\,600}$, which is given in Column 5.

The distance from the lower toe to the point on the base cut by the resultant of weight and pressure is given by Equation (13). When $\frac{w}{b}$ is 1 000, this becomes, in feet, $\frac{y\, (52\,575 + 77\, y)}{140\, (600 + y)}$, which is given in Column 6.

Similarly, the horizontal distance of the upper toe from the center of gravity is given by Equation (8). When $\frac{w}{b} = 1\,000$, this becomes, in feet, $\frac{11\, y\, (225 + y)}{20\, (600 + y)}$, which is given in Column 7.

TABLE No. 1.

y, in feet.	Volume per foot, in cubic feet.	Maximum Pressure, in Tons per Square Foot.			Distance from Toe to Center of Pressure.		Coefficient of friction, tan. a.	Factor against over-turning.
		Full.	Multiplied by sec.^{2} a.	Empty.	Full.	Empty.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
10	27.1	5.32	8.64	0.50	6.24	2.12	0.7904	3.34
20	110.2	5.47	8.78	1.03	12.47	4.35	0.7776	3.34
30	252.0	5.62	8.91	1.55	18.67	6.68	0.7653	3.35
40	455.1	5.76	9.03	2.08	24.85	9.11	0.7534	3.35
50	722.2	5.91	9.16	2.61	31.00	11.64	0.7418	3.35
60	1 056.0	6.06	9.30	3.15	37.14	14.25	0.7305	3.36
70	1 459.1	6.20	9.44	3.70	43.26	16.95	0.7196	3.36
80	1 934.2	6.35	9.57	4.25	49.35	19.73	0.7090	3.36
90	2 484.0	6.50	9.69	4.81	55.44	22.60	0.6987	3.37
100	3 111.1	6.65	9.80	5.37	61.51	25.53	0.6888	3.37
110	3 818.2	6.79	9.92	5.95	67.55	28.55	0.6791	3.37
120	4 608.0	6.94	10.04	6.52	73.59	31.62	0.6696	3.38
130	5 483.1	7.09	10.18	7.10	79.61	34.77	0.6605	3.38
140	6 446.2	7.23	10.30	7.70	85.62	37.98	0.6515	3.38
150	7 500.0	7.38	10.43	8.29	91.61	41.25	0.6428	3.39
160	8 647.1	7.53	10.56	8.89	97.59	44.58	0.6344	3.39
170	9 890.2	7.68	10.69	9.49	103.55	47.96	0.6262	3.39
180	11 232.0	7.82	10.81	10.10	109.51	51.40	0.6181	3.40
190	12 675.1	7.97	10.93	10.72	115.46	54.90	0.6103	3.40
200	14 222.2	8.12	11.07	11.34	121.39	58.44	0.6027	3.40

Column 8 gives the coefficient of friction necessary to stability. See Equation (17).

The factor of safety against overturning is $\frac{W \text{ by its arm}}{P \text{ by its arm}}$, which, with these assumptions, becomes $3.3367 + 0.0034\,y$, being always more than 3.33, and increasing slightly with the height of the dam. This is given in Column 9.

By permission of Mr. Edward Wegmann, some of his figures will be given for comparison. Where possible, his hypotheses have been adopted with this in view. $\frac{w}{b}$ has been taken as 1 000 to agree with his maximum intensity of pressure for a 200-ft. dam.

TABLE No. 2.

50-Foot Dam.	Wegmann's No. 1.	Proposed Dam.
Height	50 ft.	50 ft.
Crest	5 "	5 "
Specific gravity of masonry	2.33	2.33
Volume per foot of length	830.7 cu. ft.	722.2 cu. ft.
Maximum pressure, reservoir full	3.58 tons.	5.91 tons.
" " " empty	3.67 "	2.61 "
Coefficient of friction necessary	0.64	0.74
Coefficient against overturning	2.00	3.35

100-Foot Dam.	Wegmann's No. 2.	Proposed Dam.
Height	100 ft.	100 ft.
Crest	10 "	10 "
Specific gravity of masonry	2.33	2.33
Volume per foot of length	3322.7 cu. ft.	3111.1 cu. ft.
Maximum pressure, reservoir full	7.16 tons.	6.65 tons.
" " " empty	7.33 "	5.37 "
Coefficient of friction necessary	0.64	0.69
Coefficient against overturning	2.00	3.38

200-Foot Dam.	Wegmann's No. 3.	Proposed Dam.
Height	200 ft.	200 ft.
Crest	18.74 ft.	20 "
Specific gravity of masonry	2.33	2.33
Volume per foot of length	15195.1 cu. ft.	14222.2 cu. ft.
Maximum pressure, reservoir full	8.13 tons.	8.12 tons.
" " " empty	10.33 "	11.34 "
Coefficient of friction necessary	0.56	0.60
Coefficient against overturning	3.20	3.40

This comparison is not made in order to criticize Mr. Wegmann more than other analysts, but because his figures are reached after a digest of those of most previous writers, and show points of betterment over them. As before stated, the criticism is aimed at the assumption that, because a uniform construction to a safe profile insures a safe structure, uniformity is either necessary or economical.

The quantity of masonry in this proposed type is always less than in uniform-profile types.

The maximum intensity of pressure when the reservoir is full is more for low dams, less for high dams, but never reaches the high pressures of Mr. Wegmann's No. 3.

The maximum intensity of pressure when the reservoir is empty, up to 180 ft. in height, is less. Above that height, it is more. The vertical line from the center of gravity falls always above the middle third. The writer has never been able to see the reason for requiring it to fall inside the middle third. It does not fall dangerously near the upper toe. A foundation so yielding, or a construction so flimsy, that a wall, braced on one side with substantial buttresses, which would fail when not subjected to side pressure, is hardly to be considered in the catalogue of dam possibilities.

The factor of safety against overturning is greater in the proposed type.

DISCUSSION.

H. de B. Parsons, M. Am. Soc. C. E. (by letter).—The dam described by the author is practically a dam of uniform section reinforced by buttresses built against the down-stream face. This principle is not new, as many engineers have attempted to perfect such designs, when planning large masonry dams, in order to save yardage. As far as the writer is aware, no such buttress dam of large proportions has been constructed, as the plans have lacked the mass required for stability, or have indicated excessive toe pressures. However, exception might be made to some curved dams (such as Bear Valley) in which the abutting hillsides have replaced the artificial buttresses proposed by the author. Mr. Parsons.

In order to secure stability against sliding and overturning forces, the important point is to have mass, rather than little mass mathematically placed. The forces acting against a dam are not scientifically determinable, and, therefore, their real action is still unknown.

The usual assumption is that the masonry mass is solid without elasticity, or else solid with uniform elasticity.

With dams of uniform profile, therefore, care is taken not to transgress these assumptions, which are known to be only approximate. Such dams are calculated on their sections, each section being strong enough, *per se*, to withstand the pressures and not require any assistance from the adjacent sections.

The author appears to have altered these so-called standard assumptions by enlarging their scope to a point which may or may not be true. Thus, he figures the weight of the dam between centers of buttresses, and assumes that the weight will act through the center of gravity of the mass. The question arises, therefore, can the weight be considered as acting thus; or will not the weight of the buttresses alone be left to resist the total water pressure? This water pressure will surely come back through the buttresses on account of the arched form, but the writer doubts if much resisting weight will be available, outside of that of the masonry, along or near the axis of each buttress.

The cost of such a buttressed dam would probably exceed that of a dam having a uniform section, on account of the increased face work.

Edward Wegmann, M. Am. Soc. C. E.—The idea of constructing a dam having a number of piers joined by vertical arches is not new. This plan naturally suggests itself for saving masonry. The Belubula Dam,* constructed in South Wales about 1898, was built in this Mr. Wegmann.

* *Engineering News*, September 8th, 1898.

Mr. Wegmann. manner. This dam has a length of 431 ft. and a maximum height of 60 ft. above the foundation. The lower part of the dam is formed of a solid wall of concrete, 23 ft. high at the lowest part of the valley. On top of this wall six brick buttresses are built, 28 ft. from center to center. The piers are joined by five elliptical arches of brickwork, 4 ft. thick at the bottom and 1 ft. 7 ins. thick at the top. The arches are inclined down stream 60° from a vertical plane, and form the upper 37 ft. of the dam.

In connection with the power plant of the Pioneer Electric Power Company, of Ogden, Utah, a concrete dam 100 ft. high, consisting of a number of piers joined by arches, was proposed. The design of this dam has been fully described* by Henry Goldmark, M. Am. Soc. C. E. The dam was to consist of piers, placed 48 ft. from center to center, and be joined by cylindrical arches of concrete, 8 ft. thick at the bottom and 6 ft. thick at the top. The up-stream faces of the piers, and, consequently, of the arches, were to be inclined down stream. A roadway 16 ft. wide was to be constructed on top of the piers, and be supported on arches built between the piers. In order to insure water-tightness, the up-stream face of the dam was to be covered with ¼-in. steel plates. According to the bids received, this form of dam was found to be from 12 to 15% cheaper than an ordinary dam of uniform section. The Ogden Dam has not yet been built.

About a year ago, the speaker was engaged to design a dam 160 ft. high, which was to consist of piers joined by vertical arches. He found that he could not improve much on the design for the Ogden dam, as far as it went, viz., for a height of 100 ft., and had only to make calculations for an additional 60 ft. in height.

A French engineer has proposed to build a dam as a straight wall, having a comparatively weak section, and to reinforce it by buttresses placed at regular intervals. Some economy in construction might possibly be effected in this manner. The speaker does not know of any dam which has been originally built according to this plan, but the Gros Bois Dam, built in France in 1830 to 1838, was reinforced in 1842 by the construction of nine counterforts, as it deflected several centimeters when subjected to the full water pressure. This dam had a length of 1 805 ft. and a maximum height of 93 ft.

The new type of dam proposed by Mr. Dillman differs from the arched dams mentioned only in substituting between the piers parabolic arches for the ordinary cylindrical arches. The formulas which Mr. Dillman has evolved for his type of dam are exceedingly simple, but Mr. Dillman's type is not the most economical of its class. Applying his type to the proposed Ogden dam,

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 293.

Mr. Wegmann.

which was to be 100 ft. high and to have the piers 48 ft. from center to center, and comparing it with the design made for the Ogden dam, and, also, with a dam having a uniform cross-section, the result is as follows:

Ogden project	86	cu. yds.	per	linear	foot.
Dillman's plan	115	"	"	"	"
Uniform dam	123	"	"	"	"

This shows that, as far as strength is concerned, a considerable saving in masonry could doubtless be effected by building a dam of piers and arches.

There is, however, another consideration besides stability, *viz.*, the prevention of leakage. In the case of a high dam, even when its base is 100 ft. or more in thickness, some sweating or leakage will generally occur. How, then, could the thin face of masonry proposed by the author, which was to have only the thickness of one-tenth the height, be made water-tight? Ordinary masonry would not suffice for this purpose. Steel plates or asphalt might be used, but would add to the expense of the construction.

While steel plates would make a tight face, there would be some difficulty in connecting them at the bottom and sides of the valley so that water could not pass around the steel facing. The speaker had thought of using a facing of asphalt for a dam 160 ft. high, which was to consist of piers and arches. The trouble with asphalt, however, is that it is plastic. It could not be used for a vertical face. Experts in asphalt had advised the speaker to give the dam a big slope up stream (which was contrary to the usual design), and to imbed the asphalt between two layers of masonry. The objection to introducing in a dam asphalt or steel plates a certain distance from the up-stream face is that it breaks the continuity of the masonry, and makes the calculations of the distribution of the pressure very uncertain.

The author used the ordinary formulas for the distribution of pressure, which assume it to be either uniform or to vary uniformly, according to the position of the line of pressure. As the width of the piers of the proposed new type diminished toward the down-stream face, the author assumed the pressures to increase from what they would be for an ordinary uniform dam, in proportion as the width of the piers diminished. The formulas applied for calculating the distribution of pressure in an ordinary masonry dam are based on the assumption that the dam is perfectly rigid. While this assumption, of course, is far from being exact, it exaggerates, in all probability, the pressures near the faces of the dam where the masonry is weakest, and, therefore, errs in the direction of safety. When these formulas, however, are applied to piers like those proposed by the author, subjected not only to the water pressure and their own weight, but,

Mr. Wegmann. also, to arch strains, and having bases that diminish in width down stream, there is a great deal of uncertainty as to the results obtained. The more eccentric the lines of pressure, the less likelihood of the usual formulas for calculating the maximum pressure being applicable to the case.

The author states that he did not see the necessity of limiting the lines of pressure to the center third of the profile, a restriction usually made in the designs of dams since Professor Rankine drew attention to this matter in his valuable report* on "The Design and Construction of Masonry Dams."

According to the laws of a uniformly varying stress, upon which the formulas for the distribution of pressure in a dam are based, tension would occur in the face further from the line of pressure, whenever the latter fell outside of the center third of the profile. While it may seem a little difficult to understand how tension could occur in a wall of masonry, it is likely to take place in the case of an eccentric line of pressure, which would tend to tilt the dam, causing great pressure near one face and a tendency to open at the other.

As regards the question of practical construction, Mr. Dillman's type of dam would be rather troublesome to execute, especially if the down-stream face were to be made of cut stone, as the parabolas forming this face are continually changing according to the height of the dam. Cylindrical arches would be more easily built.

Mr. Dillman has stated that no more intricate mathematics would be involved if the up-stream face of his new type of dam were battered instead of vertical. That might be the case if the weight of water resting on the up-stream face were not considered in the calculations. This omission might be safely made if the up-stream face were steep, as the error involved would be trifling, and in the direction of safety. If the up-stream face, however, had considerable batter, and was similar, for instance, to that assumed for the design for the Ogden Dam, the weight of the water resting on this face, and its moment, would have to be included in the calculations, and, doubtless, would make the formulas for determining the cross-section of the dam more complicated.

If some good method were discovered for making a thin wall of masonry practically water-tight under considerable pressure, dams consisting of piers joined by vertical arches would be likely to be constructed in the future, with a view of saving expense. In our present state of knowledge, however, the design of such structures would have to be based upon the results obtained by actual construction, low dams being built at first, and others of greater height when the lower dams had stood successfully for some years.

Mr. Fuller. WILLIAM B. FULLER, M. Am. Soc. C. E.—A dam of the design proposed by the author is worthy of careful study, and such study would

* *The Engineer*, January 5th, 1872.

lead to a better realization of the essential factors which enter into the construction of dams. There are one or two details of this design which seem very favorable. The leakage through the dam could be under inspection, and controlled so that exactly what was occurring at any time would be known. If there is only a very thin sheet in front, and that can always be inspected, it would be known whether there was anything going on which required repairing or taking care of, and it would seem that this is a matter which ought to be known about any dam, rather than to know hardly anything about it, as in the so-called gravity type. Mr. Fuller.

There should not be any more difficulty in making a dam tight with masonry a few feet thick, say 10 ft., under 100 ft. head, as proposed by the author, than there would be in making it tight if the masonry were 40 to 80 ft. thick; in fact, it would probably be easier to make the thin wall tight than the thick one.

The speaker's experience has been that, where there is a thin sheet of masonry, a great deal more attention can be paid to the detail of its construction, and a great deal richer proportion of cement used with the concrete or masonry without incurring prohibitive costs of construction. In this way it would be possible to get a very much higher standard of construction than is ordinarily obtained when a large, thick masonry dam is built.

The speaker is of the opinion that there will not be very much difference in the cost of construction in either case ; that is, the cost would be put into masonry of a better quality.

The discussion has apparently turned on the fact that such a dam would be of cut-stone masonry, and therefore very expensive to construct, owing to the large amount of face work. In these days it is hardly probable that any such dam would be built ; it would probably be of concrete, which seems to be the favorable construction material at present, and it would not cost much more per unit to make a thin section of such material than a very heavy one.

About three years ago the speaker had occasion to design a dam to hold about 43 ft. of water, and made three different designs. One was a gravity section, of concrete ; another was a section somewhat similar to that proposed by the author, with the exception that it had two faces and the interior was hollow, the idea being that a man could pass through the interior of the dam and inspect it at all times, and any seepage through the material would be instantly located. In order to get stability for such a design, it was necessary either to make a very wide base or put in iron, and in this design iron was used to take up the additional stresses instead of using a wide base. Another design considered the dam merely as an iron cantilever truss, entirely surrounded with concrete, and with an apron of concrete in front held up by this truss, thus being very similar in design to that proposed by

Mr. Fuller. the author. Bids were received on all three of these designs, and the gravity section being very much the cheapest was therefore adopted and built.

Mr. Pegram. George H. Pegram, M. Am. Soc. C. E.—The form of dam proposed by the author aims at a saving of masonry, but its main advantage seems to be in the marked increase in overturning resistance shown in his table of comparisons with the Wegmann forms. It also has the advantage claimed by the author of greater stability against the upward pressure of water leaking under the dam.

The type proposed seems susceptible of very neat calculations, but this is not an important consideration in dams of magnitude.

The Pioneer Electric Power Company, of Ogden, Utah, of which the speaker was Consulting Engineer, studied the question of a 100-ft. dam for the Ogden River, and some of the results are given in a paper by Henry Goldmark, M. Am. Soc. C. E.*

At the time the speaker's attention was called to the matter, the company had prepared designs for the ordinary gravity section and also for a steel dam with a vertical face, and it was suggested that, in any dam where the weight of masonry is reduced, the up-stream face should be inclined so as to use the weight of the water, and it is thought that this suggestion might apply to the form submitted by the author of the paper under discussion.

In the case of the Ogden Dam the walls of the cañon are so precipitous that a diversion of the river seemed a serious question, and the location of a city at the mouth of the cañon made it especially desirable to design a dam which would be as absolutely safe as due considerations of cost would permit. The speaker suggested that a form be tried having isolated piers connected by arches with impervious faces, a form of which is given in Mr. Goldmark's paper; the objects to be accomplished being:

1.—The construction of piers independently, thus avoiding the necessity of a diversion of the river;

2.—The securing of an impervious face to the dam;

3.—The avoidance of uplifting pressure by water that might leak under the dam;

4.—The ability to get a better class of masonry by building it in smaller masses than in the ordinary dam;

5.—Less total cost.

Cracks in concrete are apt to be long, and hard to stop, and therefore the necessity of a steel plate on the up-stream face. Large steel plates, at the time, could have been bought at a trifle more than one cent per pound, and the speaker proposed a design having isolated piers of triangular form with an up-stream slope of 30°, 11½ ft. thick, and spaced 23 ft. in the clear, the up-stream face being a continuous

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, pp. 290 and 302.

steel plate, corrugated with radii of 7 ft. over the piers' ends and 14 ft. between the piers, the up-stream toe to be enclosed in a wall of concrete, 15 ft. thick and 25 ft. high, enclosing horizontal angle-irons riveted to the steel plate to assist by its weight in resisting the overturning effect. Mr. Pegram.

Mr. Breuchaud. J. Breuchaud, M. Am. Soc. C. E.—It is somewhat difficult to give an off-hand estimate of the cost of the masonry in the author's dam as compared with that in a dam of the ordinary section; but the speaker believes the cost would be greater, because it would require a high grade of work, with deep, close-cut beds and joints, to ensure anything like a reasonable degree of water-tightness in such a thin wall as that proposed.

Mr. Abbott. A. V. Abbott, M. Am. Soc. C. E.—The speaker has had little experience in the actual construction of masonry dams, but has had considerable in the endeavor to construct masonry in such a manner as to make it completely water-tight; and so it appears to him that the points made by Mr. Pegram are exceedingly well taken.

There are two aspects of engineering: One, the theoretical point of view, and the other, the adaptation of the design indicated by theory to practical conditions. There appears to be little doubt that a dam or other structure intended to enclose water could be designed with a more economical section, upon the principles developed in this paper and discussion, than according to the ordinary methods. Doubtless there would be a considerable saving in the actual quantity of masonry; but when the problem is presented of making water-tight a structure as thin as would be called for if the principles of this paper were followed out, a very difficult problem is introduced, and one for the solution of which quantity of masonry rather than strength of construction is required. It also seems that the subdivision of a dam into a number of buttresses, as is demanded by the plan proposed in the paper, is likely to introduce such a complication of stresses as will make accurate determination thereof, even theoretically, an exceedingly difficult problem, and one which is entirely incapable of solution under ordinary practical conditions.

Mr. Dillman. George L. Dillman, M. Am. Soc. C. E. (by letter).—Discussion has developed the following criticisms:

1st. The mass is not sufficient.—Mr. Parsons says that this is on account of stability against sliding and overturning; Mr. Abbott says it is on account of difficulties in making the dam tight.

According to the generally accepted laws of friction (which are wrong, but which need not be discussed here), any decrease in mass decreases the factor against sliding, but it is distinctly against elementary engineering to say of stability against overturning: "The important factor is to have mass, rather than little mass mathemat-

Mr. Dillman. ically placed." If mass alone is to be considered, what difference does the shape make? The "standard types" might be built upside down.

Tightness is considered further on.

2d. The stresses are not determinable.—The stresses may not be measurable, but they are more nearly determinable than stresses in a uniform-sectioned dam, because the point where the water is cut off is more nearly known.

The internal stresses caused by the setting of the concrete masonry are probably not different from those in a uniform-sectioned dam. Were these large enough for danger, it is certain that each buttress would have to stand for itself, as the thin part of the wall would transmit no large strain to the next buttress.

As for external forces, weight will act through the center of gravity more certainly than in a dam in which water penetration is unknown. The same can be said of the center and direction of water pressure. It should be noted that the unit can be considered from center to center of arches as well as from center to center of buttresses.

3d. The plan is difficult to execute and expensive to build.—This criticism is made most frequently by those who speak of a cut-stone face. The writer has no idea of using cut stone or even coursed masonry in a dam, except for coping and trimming. Rubble and concrete are the materials for such work to-day. Even rubble is getting to be inexpedient, except to save cement when the haul is expensive.

The curved surface can be developed into a plane, so that either the frames or the lagging of the forms can be straight lumber. For dams of considerable size, frames should probably be up and down, and the lagging bent to these straight frames. For lower dams, when the curvature is sharper, probably horizontal frames with up and down lagging might be better. In either case, the cost is not excessive, and the parabolic is as cheap as any other curved form.

The writer is engaged now in putting in some concrete dam work in which he is using cylindrical, truncated-conical, elliptical and parabolic forms, and, except for a few minutes with his carpenter foreman on each new shape, cannot see that there is any trouble, or much difference in cost between them and straight forms. There is no more lumber used than would be used in a broken surface approximating the curve. The cost of band-sawing the frames is insignificant, and a yard of concrete is rammed against a curved form as cheaply as against a straight one.

4th. A thin wall cannot be made tight.—Mr. Fuller has replied to this criticism by saying: "It would probably be easier to make the thin wall tight than the thick one." That has also been the writer's experience.

There is another consideration: If a dam cannot be made tight at its up-stream face, it is a mistake to tighten it at all. The up-stream

face, if not tight, should be the most nearly tight surface. All water passing it should be allowed to escape without exerting further pressure. The penetration of water to some tighter surface is the principal source of doubt in dam stresses. Such penetration practically shifts the center of gravity and the direction of water pressure. Mr. Dillman.

5th. It is not the most economical of its class.—No such claim was made; but the Ogden Dam, with a steel face anchored to bed-rock or massive concrete to stand tension, is hardly in the same class. Economy is not the best point in the proposed type. Increased factors of safety, and chance for inspection and repair are of much more importance.

The dimensions, batters, curves, specific gravity of the masonry, and the allowable strain used, were adopted for purposes of fair comparison, simplicity and the avoidance of certain criticisms. The general type of buttressed wall can be cheapened materially beyond the particular one analyzed.

6th. It is not new.—It seems to the writer that finding out what has been done, and, therefore, with certainty what can be done, is fully as important to successful engineering as new discoveries. Besides, it was absolutely new to the writer when, in the prosecution of work, he discovered it. It seemed new to some of his friends to whom he showed it. It could not have been "readily found elsewhere," or the Publication Committee of the Society would not have published it.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 938.

THE FLOW OF WATER IN WOOD PIPES.*

By Theron A. Noble, M. Am. Soc. C. E.

With Discussion by Messrs. E. W. Schoder, A. V. Saph, Mansfield Merriman, Rudolph Hering, Gardner S. Williams and Theron A. Noble.

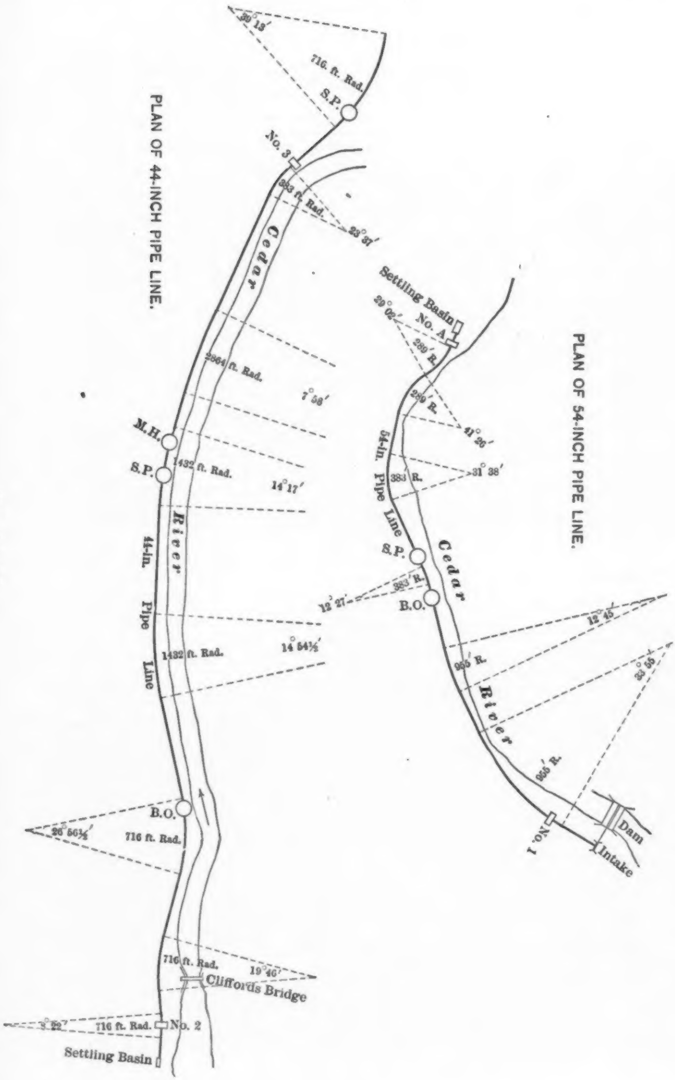
Before entering into a discussion of the series of experiments conducted by the writer, through the courtesy and assistance of Reginald H. Thomson, C. E., City Engineer of Seattle, Wash., a brief description of Seattle's gravity supply will be required, in order to fully understand the conditions under which the experiments were conducted.

Description of Gravity System.—About 24 miles southeast of Seattle, as the crow flies, the city has constructed, across Cedar River, a dam which diverts the water for the city's domestic water supply into a series of pipe lines, 28.52 miles in length, connecting the dam directly with the high-service reservoir in the city. These lines consist of 54-in., 44-in. and 42-in. stave-pipe, and 42-in. steel pipe.

The first section consists of 2 679 ft. of 54-in. stave-pipe extending from the intake at the dam to the settling basin. The plan and profile of this section are shown in Figs. 1 and 3.

The dam and the upper part of the 54-in. pipe line are shown in Figs. 1 and 2, Plate I.

* Presented at the meeting of October 1st, 1902.


PLAN OF 44-INCH PIPE LINE.
PLAN OF 54-INCH PIPE LINE.
Cedar River
44-in. Pipe Line
54-in. Pipe Line
Settling Basin
No. 2
No. 3
No. A
No. 1
S.P.
M.H.
B.O.
Cliffords Bridge
Dam
Intake
716 ft. Rad.
383 ft. Rad.
2864 ft. Rad.
1432 ft. Rad.
289 R.
383 R.
955 R.

FIGS. 1 AND 2

At the intake, the water is controlled by a large sluice-gate, operated in the usual manner, with racks, pinions and handwheel.

At the settling basin, Fig. 4, the pipe enters a well where the water is either diverted into a 24-in. by-pass, by shutting down a large wooden sluice-gate, or allowed to flow through the basin. The opening between this well and the basin is 5 ft. in diameter, and is built in the concrete wall dividing the well from the basin.

The water entering the settling basin passes through two sets of screens extending across the basin and through a 48-in. sluice-gate into the 44-in. pipe, the first of a series of 44 and 42-in. wood and steel pipes leading direct to the high-service reservoir in the city.

There are only two connections with this line; one 24-in. connection leading to Beacon Hill Reservoir (which was the reservoir used in connection with the old pumping system), and one 36-in. connection leading to the low-service reservoir. Neither of these connections is in use at the present time.

The settling basin is 80 ft. long (including the well), 25 ft. wide and 18 ft. deep. It is intended for separating any foreign matter which may pass through the racks at the intake, and is arranged with a by-pass, so that it is not necessary to shut down the line while the basin and screens are being cleaned. It is drained through three 24-in. sluice-gates, which discharge into three cast-iron pipes connecting with

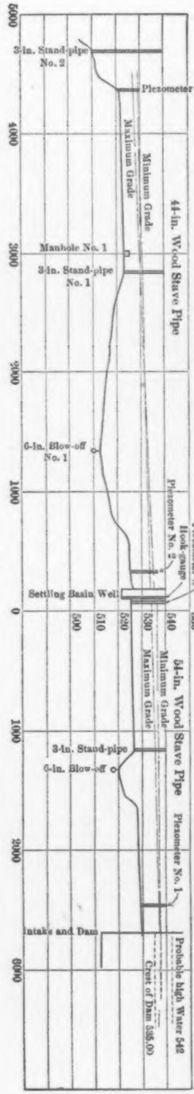


FIG. 3.

PROFILE SHOWING LOCATION OF PIEZOMETERS AND HYDRAULIC GRADE LINES.

FIG. 1.—DAM AND INTAKE, SEATTLE WATER-WORKS.

FIG. 2.—UPPER PART OF 54-IN. PIPE, SEATTLE WATER-WORKS.

a
th
su
q
va
th

tu
3
p
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a common drain tunnel. There is also a 3-in. valve connecting with the 24-in. by-pass, and a drain tile connecting with a subsoil drain surrounding the foundation, which also drains into this tunnel. The quantity of water coming from this drain tile and from the 3-in. valve was so small that the proportion coming from outside the basin, through the drain tile, has not been taken into account. The drain

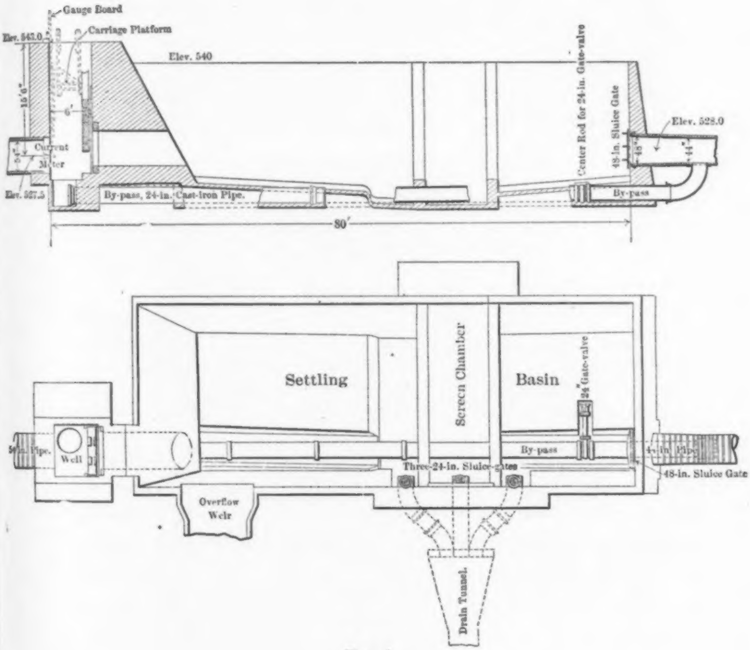

FIG. 4.

tunnel is rectangular in section, with an arched roof built of concrete, 3 ft. wide and 5 ft. high inside. During these experiments it was impossible to prevent leakage through these drain valves. The quantity was measured with a weir at the mouth of the drain tunnel, and allowed for. From the settling basin the water passes into the following pipes consecutively:

71 262 ft.	of	44-in. stave-pipe;
5 405	"	42-in. stave-pipe;
8 982	"	42-in. steel pipe across Black River Valley, where the maximum pressure is about 470 ft.;
13 503	"	42-in. stave pipe;
16 545	"	42-in. steel pipe, crossing Dunlap's Cañon;
18 163	"	42-in. stave pipe;
6 385	"	42-in. steel pipe crossing a low divide in the hill above the city;
7 687	"	42-in. stave pipe to the high-service reservoir.

Making in all:

1 section	of	2 679 ft.	of	54-in. stave pipe.
1 "	"	71 262	"	44-in. "
4 sections	"	44 758	"	42-in. "
3 "	"	31 912	"	42-in. steel pipe.
Total....		150 611 ft., or 28.52 miles.		

The following are the principal elevations:

Crest of dam	536	ft.
Crest of weir at high-service reservoir	421.33	"
Total friction head	114.67	ft.
Average per 1 000 ft	0.761	"

It had been the intention, originally, to take such measurements as would make it possible to determine the loss of head in each of the different kinds of pipe, but, owing to the impossibility of measuring the discharge at any point in the line except at the head works, and the fact that there was considerable leakage at several points along the line, the writer concluded to confine his experiments to the two sizes of pipes near the head works, *viz.*, the 54-in. pipe leading from the dam to the settling basin, and the 44-in. pipe leading out of the settling basin for a distance of about 4 200 ft. These two lengths being nearest the point of measurement, and under comparatively light pressure, there would be less danger of leakage, and it was possible to measure the loss of head with open piezometer tubes.

Location of Piezometers.—The question of the proper location of the connections with the piezometer tubes involved a study of the probable conditions affecting the results of the experiments.

It was desirable, in the first place, to make the distance between

PLATE II.
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IN WOOD PIPES.

FIG. 1.—PIEZOMETER No. 1.

FIG. 2.—PIEZOMETER No. 2.

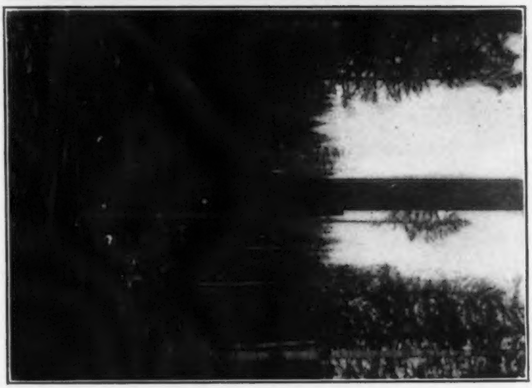

FIG. 3.—PIEZOMETER No. 3.

piezometers as great as possible, in order to reduce to a minimum errors in piezometer heights from various causes. It was of importance to locate the lower piezometer on the 44-in. pipe so that the widest range of hydraulic grade could be obtained. It was also desirable to locate the piezometers where the pipe was of uniform section, and nearest the average diameter. This, however, was a blind guess, as it was impossible to make an examination of the inside of the pipes at that time. The writer was fortunate in this respect, as, by an examination made two months later, it was found that the piezometers could not have been placed to much better advantage.

Piezometer No. 1, the upper one on the 54-in. pipe, was located 232 ft. from the intake. (See Figs. 1 and 3, and Plate II, Fig. 1.)

The lower piezometer heights, for the 54-in. pipe, were taken in the well with a hook-gauge, Fig. 5. This well is 5 x 7 ft., and 19 ft. deep inside. The measurements for flow were taken with a current meter, from a platform suspended in the well above the opening from the pipe.

HOOK GAUGE.

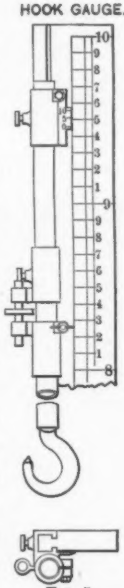

Fig. 5.

After taking the first series of observations on the 54-in. pipe (from 1 to 11, inclusive), the writer had some suspicion that the hook-gauge reading did not give the true piezometer height, and decided to attach a piezometer some distance back of the well and take two observations, one at the lowest and one at the highest hydraulic gradient used in the experiments. This piezometer (Piezometer *A*, see Figs. 1 and 3) was located 47.5 ft. from the inside face of the well.

The hook-gauge was made with brass fittings, the rod being a seamless drawn-brass tube. The hook was of steel, with the point hardened. The gauge was fastened to the well curb. The hook extended into a wooden box which protected the surface of the water from the fluctuations prevailing in the well. As the temperature during the experiments did not vary more than 10°, the changes in the length of the rod were not sufficient to be taken into account.

Piezometer No. 2 was located on the 44-in. pipe, 150.6 ft. down stream from the inside wall of the basin. (See Figs. 2 and 3, and Plate II, Fig. 2.)

Piezometer No. 3 was located on the 44-in. pipe, 4 041 ft. down stream from Piezometer No. 2, on the crest of the bank above the river valley. (See Figs. 2 and 3, and Plate II, Fig. 3.)

Description of Piezometers.—To determine what resistance, if any, there was in the fittings, in the lengths of pipe experimented upon, an oil differential gauge, Fig. 6, had been constructed to determine

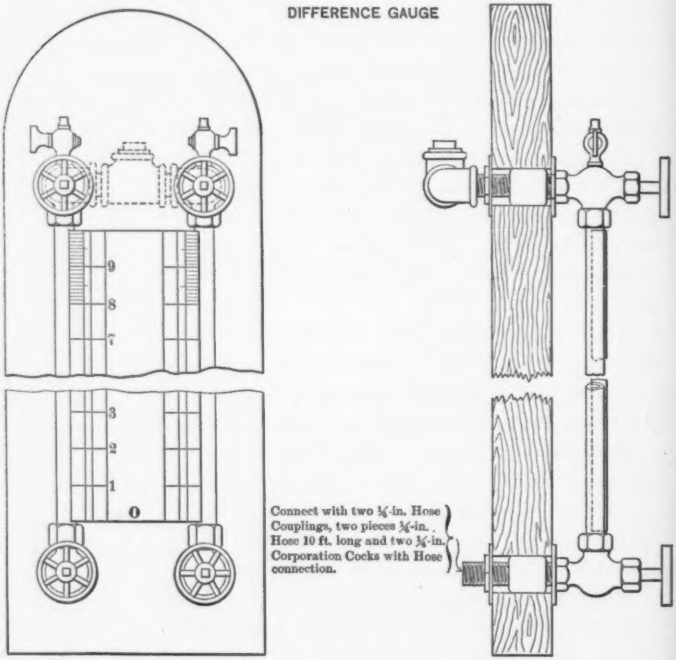

FIG. 6.

very slight differences of pressure. One side of this gauge was detached, and the glass tube and fittings were attached to another board with a separate scale. As only two piezometers were required at any one time, these two tubes answered for all purposes. The glasses were $\frac{5}{8}$ in. outside diameter, 5 ft. long in the clear, mounted in common, boiler-gauge fittings, on a $1\frac{1}{4}$-in. board. Each lower fitting was connected with 10 ft. of $\frac{1}{2}$-in. hose. The hose was connected to a

PLATE III.
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FIG. 1.—BLOCK, SHOWING HOLE BORED FOR ATTACHMENT OF PIEZOMETERS.

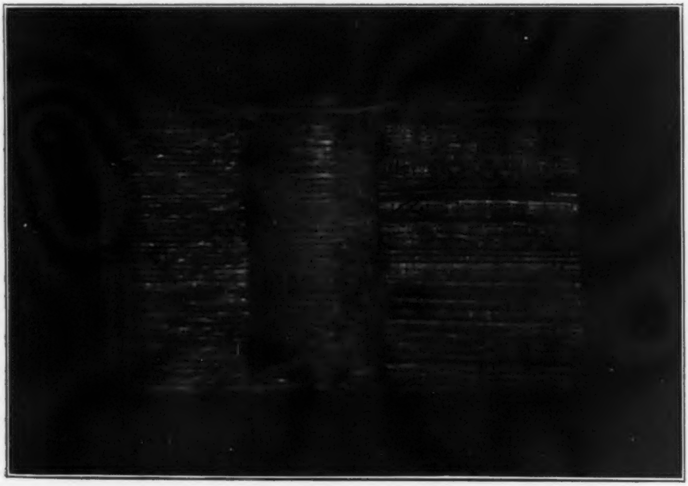


FIG. 2.—BLOCK, SHOWING HOLE BORED FOR ATTACHMENT OF PIEZOMETERS.

½-in. corporation cock screwed into a suitable hole bored into the wood pipe.

Attachment of Piezometers.—Instead of boring the hole (for inserting the corporation cock) completely through the pipe, and thus making a sharp and splintered edge, the bit was withdrawn as soon as the screw had passed through into the interior of the pipe. This left a clean hole, about $\frac{3}{16}$ in. in diameter, entirely free from any internal projection. (See Plate III, Figs. 1 and 2.) These holes were afterward examined on the inside of the pipe, and found to be free from any projection.

All holes were bored with a gauge which kept the bit perpendicular to the outer surface of the pipe.

Levels of Piezometers.—As it was not possible to shut down the gravity system, except from the head works, it was impossible to ascertain the static head at the several piezometers, except by leveling.

To determine the heights of zero on each piezometer scale, levels were run by three different observers, and the average of the two which agreed most nearly was taken as correct. The greatest difference between any two observers was 0.015 ft. The probable error is not more than 0.007 ft. The least hydraulic gradient experimented upon was 0.342 ft., so that the greatest possible error from this cause was about 2 per cent.

Measurement of Flow.—There was no choice as to the method to be used for measuring the discharge. No provision for metering the water was made when the system was planned, except a sharp-crested weir which had been provided in the gate-house of the high-service reservoir at the discharge end of the line.

In any case, this weir would not answer for measuring the discharge at the intake end of the line, where it was necessary to carry on the experiments in order to get as wide a range of friction head as possible, as the quantity of leakage in the long length of pipe was an uncertain factor.

The measurements for discharge were made with Haskell's current meter *C*, having a 7½-in. wheel, with the ordinary holding rod 15 ft. long, and the regular electrical connections, provided with a telegraph sounder. Fig. 4 shows the well. The general arrangements for holding the meter in position are shown in dotted lines.

A platform was hung in the well just above high-water level, leaving, next to the wall, an open space containing the pipe opening, about 1 ft. wide. On this platform was laid a track of two angle irons planed on their upper edges. This track was carefully leveled, and laid parallel to the face of the wall and perpendicular to the axis of the pipe. For the track, a carriage was constructed to which was attached a framework to hold the meter rod in position and allow the desired vertical and horizontal movements of the meter—the horizontal movement being obtained from the motion of the carriage on the track, and the vertical motion from the movement of the meter rod in a vertical groove in the front of the frame.

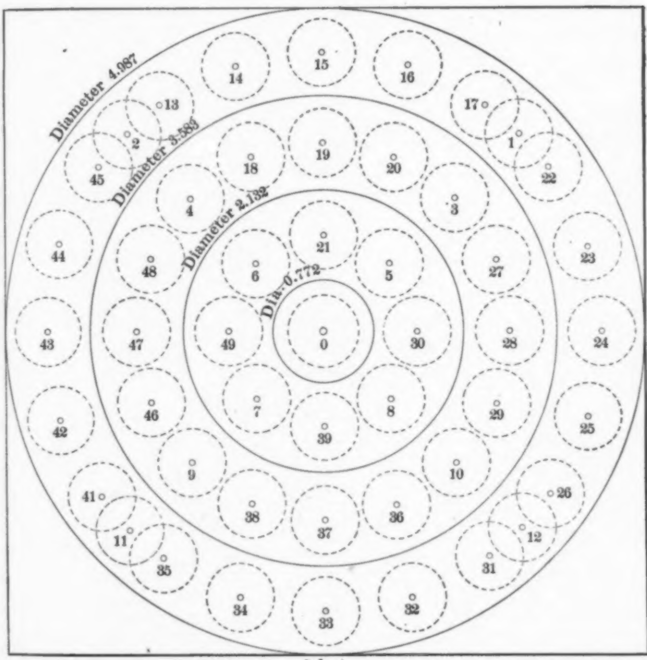
It was found that the carriage could not be made sufficiently small to give the amount of travel necessary to get the two extreme velocity readings of the horizontal diameter of the opening, thus covering only 47 out of 49 readings for the whole area of the opening. To the top of the meter rod was clamped a block of wood, 6 ins. wide and 18 ins. long, near the top of which was a hole for inserting a pin used to hold the meter and carriage in position on the board, while taking each velocity reading. The front face of this block was placed on the meter rod exactly at right angles to the axis of the meter wheel, so that, when held in position against the guide board, the axis of the wheel would be parallel to the axis of the pipe.

The guide board, Fig. 7, was made of 1-in. D. and M. flooring, 5 ft. square, braced on the back and fastened to the curb of the well in such a position that, when the wheel was in the center of the opening, the pin in the top of the block would be approximately in the center of the board. A blank wheel, 1 in. larger in diameter than the meter wheel, was then put on the spindle of the meter, and the meter, with the rod, etc., was put into place. The blank wheel was then brought up against the inside surface of the opening at various points on its circumference, and the corresponding points on the board were plotted from the hole in the block. Through these points a circle was drawn, which was the line for the outer set of holes to be used in locating the meter for the velocity readings.

From this circle, the outside circumference of the pipe, as projected on the board, was located, and the center and four diameters were drawn. This projected area of the opening was then divided into four concentric rings (including the center), and a circle was

drawn in the center of each ring; the holes for the velocity readings being all spaced on these circles, the velocity readings would be taken in the center of each ring. The average of the velocity readings inside of each ring was calculated separately and multiplied by the area of the corresponding ring, to determine its discharge, the sum of the discharges of the separate rings making the total discharge through the opening.

GUIDE BOARD



5 feet

5 feet

FIG. 7.

The spacing of these holes on the circles was first made on four diameters, and the intervening spaces between the diameters were filled in at regular intervals, making in all 47 holes, not counting the two holes at the ends of the horizontal diameter, which could not be used.

The area of the opening is 19.533 sq. ft., which is not the same as the area of the end of the pipe (16.605 sq. ft.), as the pipe does not extend to the well, within 8 ins., but is the same as the outside area of the pipe.

Before everything was in working order, it was found that some part of the frame or platform had sprung sufficiently to misplace the meter about 1 in., so that when the pin was placed in the outer holes on the left side of the guide board the meter would foul, making it necessary to bore several new holes on this side nearer the center of the circle. This misplaced all other holes 1 in. It was assumed that this would not affect seriously the general average of the discharge measurements; it was, in fact, an advantage to have the velocity readings taken closer to the left of the opening, as the velocity was higher on this side on account of a curve to the right about 50 ft. back of the well.

The current meter was rated both before and after the experiments. The rating before the experiments was not very satisfactory, on account of surface currents. The second rating was made under more favorable conditions, and the boat was run twice in opposite directions over the same course at each velocity.

Two minutes were occupied in each velocity reading at the well: One minute for counting the sounder and one for changing the position of the meter rod to the next hole.

In order to determine whether it would be necessary to take readings from all the holes on the guide board, an observation was taken first at the lowest velocity, with a reading at every hole on the guide board (with the exception of the two at the extremities of the horizontal diameter). The order of taking readings was as follows: 1, 3, 5, 0, 7, 9, 11; 2, 4, 6, 0, 8, 10, 12; 15, 19, 21, 0, 39, 37, 33; 47, 49, 0, 30, 28; 13, 14, 18, 16, 17, 20, 22, 23, 27, 25, 26, 29, 31, 32, 36, 34, 35, 38, 46, 41, 42, 45, 44, 48.

This order of taking readings was maintained in all subsequent observations.

The discharge at the opening from all, or 50,
readings was .... 36.529 cu. ft. per second.
From two diameters, or 14 readings (1, 3, 5,
0, 7, 9, 11; 2, 4, 6, 0, 8, 10, 12), was..... 36.621 " "
From one diameter, or 7 readings (1, 3, 5, 0,
7, 9, 11), was .... 36.614 " "

From one diameter, or 7 readings (2, 4, 6, 0,
8, 10, 12), was .... 36.644 cu. ft. per second.

A difference of 0.2% in favor of two diameters.

A similar comparison was made in Observations 5 and 6 on the 44-in. pipe:

From all, or 50, readings.................... 43.059 cu. ft. per second.
From two diameters, or 14 readings........ 43.355 " "

A difference of 0.7% in favor of two diameters.

In Observations 9 and 10, on the 44-in. pipe:

From all, or 50, readings.................... 49.655 cu. ft. per second.
From two diameters, or 14 readings .... 50.089 " "

A difference of 0.86% in favor of two diameters.

The percentage of difference would seem to increase with the increase of velocity of discharge. As these percentages are within the limit of probable error of the meter, no conclusion in this respect can be drawn.

All other observations were taken from the two diameters, 1, 3, 5, 0, 7, 9, 11 and 2, 4, 6, 0, 8, 10, 12, drawn 45° from the horizontal. The conditions under which these experiments were conducted did not make it possible to take the time necessary to take readings from every hole in all the 22 observations.

Observation 4 was a repetition of Observation 3, on the 54-in. pipe, the readings for velocity being taken from two diameters in each case.

The discharge in these two observations was 49.222 and 48.869 cu. ft. per second, respectively—a difference of 0.7 per cent.

Plan of Conducting Experiments.—Each set of experiments required readings to be taken at three points. In the experiment on the 54-in. pipe, one observer was stationed at Piezometer No. 1, and two in the well, one for taking readings for the meter and one at the hook-gauge. One assistant was required for manipulating the meter.

The meter readings and hook-gauge readings were taken every two minutes and the piezometer readings every minute. No attempt was made to take the readings of the different piezometer heights at the

same instant, though they were all taken during the same interval of time. Before beginning each observation, the watches were compared and put approximately together.

The plan for the 44-in. was the same as for the 54-in. pipe experiments, except that the hook-gauge readings, in the well, were no longer necessary, and were only taken occasionally, to determine the amount of fluctuation, if any, in the height of water in the basin, and to check the height of Piezometer No. 2. One observer attended to this and also took occasional readings of head on the waste weir.

Regulation of Flow.—The minimum flow in each case was regulated by the normal capacity of the gravity system. No material increase in the maximum flow could be made without drawing the hydraulic grade line below some of the summits, located from 4 to 15 miles below the settling basin. The height of water could be reduced on the highest summit about 5 ft. This height determined the maximum discharge, in each pipe, that could be obtained during the experiments.

The flow was regulated, during the 54-in. pipe experiments, from one of the 24-in. sluice-gates draining the basin.

In the 44-in. pipe experiments, the flow was regulated by opening two 6-in. blow-offs located in the valley a short distance below Piezometer No. 3, it being found necessary to open two blow-offs in order to get the maximum flow allowable.

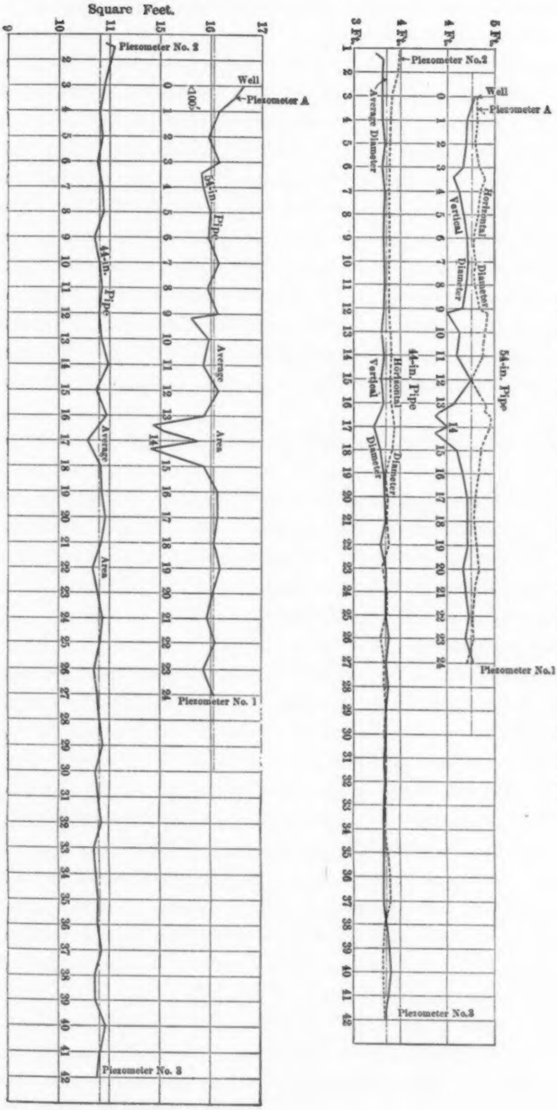
During Observations 1 to 7, inclusive, the 48-in. sluice-gate, at the entrance of the 44-in. pipe, was open about 9 ins. The sluice-gate was kept partly closed, in order to bring the hydraulic grade within the level of Piezometer No. 3.

During Observations 8 to 11, inclusive, the sluice-gate was wide open.

Size and Condition of Pipes.—Until two months after the experiments were made, when that portion of the pipes experimented upon was carefully examined, nothing was known of the condition of the interior. The writer was assured that there were no appreciable distortions, and that the pipe had been measured at intervals by the inspector, and found to be all right.

To be able to have some definite knowledge of the condition of the interior, and determine the average size, beyond question, the writer, with one of the city draughtsmen, Mr. Plachy, measured, at regular intervals of 100 ft., the vertical and horizontal diameters. Dimen-

PROFILE SHOWING DISTORTION IN PIPE LINES EXPERIMENTED UPON.



FIGS. 8 AND 9.

sions were also taken in the 54-in. pipe, at some intermediate points where the distortion was excessive.

The interior of the 54-in. pipe was badly distorted in places, as is shown in Fig. 8, the profile of the vertical and horizontal diameters being shown. This distortion is accurate only at each 100-ft. station.

Fig. 9 shows the profile of the area. The average diameter was calculated by adding the averages to the vertical and horizontal diameters of each station and dividing by the number of stations.

The areas calculated from the average of the vertical and horizontal diameters is equivalent to the area of an ellipse of the same diameters. The assumption that the distortion is in the form of an ellipse is only approximately correct, as the top and bottom assume a flatter shape when distorted by earth pressure. The error from this cause would be very slight.

There was no rupture in the pipe, due to distortion, except at one point where the pipe evidently rested on a stone which had bulged in the bottom of the pipe, for the width of two or three staves, and splintered one stave slightly on the inside. Judging from the appearance of the interior, the writer does not believe that there was any considerable leakage due to distortion, and, from the nature of the ground, any leakage that would affect the results would be seen from the outside in the form of springs.

The maximum difference between the horizontal and vertical diameters at any one station in the 54-in. pipe was 1.187 ft.; the minimum average diameter at any one station, 4.343 ft., and the maximum average diameter at the well, 4.605 ft.

The maximum difference between the vertical and horizontal diameters in the 44-in. pipe was 0.448 ft.; the maximum average diameter at any one station, 3.736 ft.; the minimum, 3.672 ft.

There was some growth on the interior of the 54-in. pipe, in the nature of *Spongilla*. The whole interior surface, with the exception of the bottom, had scattering bunches of this little sponge-like substance, which required something of an effort to remove. They projected from the interior surface of the pipe about $\frac{3}{16}$ in., and each occupied a space of about $\frac{1}{4}$ sq. in. This growth seemed to be thickest on the top, and was absent on the bottom. The 44-in. pipe was entirely free from any growth. The interior surface was very smooth, and the

distortion very little, except in comparatively soft ground—indicating lack of care in tamping.

The pipes had been in use about one year when this examination was made. That this growth should be in the 54-in. pipe and not at all in the 44-in. pipe is, perhaps, due to the difference in velocity; in the former pipe the velocity being 2.282 ft. per second, and 3.464 ft. per second in the latter, under nominal conditions. It is not likely that the *Spongilla* could get lodgment as readily with the higher velocity, or, it may be that the settling basin, being a break in the continuity of the pipe, prevented them from spreading to the 44-in. pipe.

There is no question that this growth will, in time, reduce the discharging capacity of the 54-in. pipe, and more especially will it reduce the discharge of the smaller pipes, should it get lodgment in them.

Description of Tables.—Table No. 1 shows the principal data from the 54-in. pipe experiments.

Observations 1 to 11, inclusive, were taken to determine the loss of head and the velocity of flow, within the greatest range of level in the basin that it was possible to obtain.

In Observations 1, 3, 4, 5, 6, 7, 8, 9 and 10, the piezometer heights are the average of 29 readings, with 1-minute intervals, and the current-meter readings were taken on two diameters, each reading for velocity being taken for 1 minute, with 2-minute intervals. The hook-gauge height is the average of 15 readings, with 2-minute intervals.

Observation 2 was taken in the same manner, except that there were 50 hook-gauge and meter readings and 101 piezometer readings.

Observations 12 and 13 were taken without current-meter measurements, and with 15 readings each of Piezometer No. 1, Piezometer *A* and the hook-gauge.

Columns 5, 7, 9 and 11 show the sums of the average readings in Columns 4, 6, 8 and 10, respectively, and the elevations given in the heading, and are the average elevations of the piezometer heights.

Section 1 includes the connections between the water above the dam and Piezometer No. 1. Section 2 includes the pipe between Piezometer *A* and the well. Section 3 includes the pipe between Piezometer No. 1 and the well.

TABLE No. 1—(*Continued*).

	Loss of Head. Section 3. Length, 2 446.74 ft.			Velocity, in Feet per Second.					Discharge, in Cubic Feet per Second.				
Number of observation. (1)	Difference of level from Piezometer No. 1 to well. (18)	Difference of level from Piezometer No. 1 to well, plus increase at well. (19)	Net loss of head per 1 000 ft. (20)	V_1, outer ring. (21)	V_2, second ring. (22)	V_3, third ring. (23)	V_4, center ring. (24)	Average: 54-in. pipe. (25)	Outer ring: Area = 9.450 (26)	Second ring: Area = 6.493 (27)	Third ring: Area = 3.181 (28)	Center ring: Area = 0.409 (29)	Total discharge. (30)
1........	0.834	0.836	0.342	1.544	2.183	2.191	2.167	2.282	14.591	14.174	6.970	0.886	36.621
2........	0.835	0.837	0.342	1.536	2.175	2.199	2.191	2.276	14.515	14.122	6.995	0.896	36.529
3........	1.066	1.068	0.436	1.795	2.539	2.563	2.523	2.650	16.965	16.486	8.153	1.032	42.534
4........	1.358	1.366	0.558	2.069	2.936	2.960	2.912	3.067	19.552	19.033	9.416	1.191	49.222
5........	1.356	1.363	0.557	2.221	2.669	2.936	2.960	3.045	20.989	17.330	9.339	1.211	48.869
6........	1.634	1.645	0.672	2.298	3.268	3.276	3.267	3.408	21.716	21.219	10.421	1.336	54.692
7........	1.900	1.915	0.783	2.539	3.534	3.574	3.574	3.724	23.994	22.946	11.369	1.462	59.771
8........	2.076	2.094	0.856	2.669	3.727	3.776	3.785	3.924	25.222	24.199	12.011	1.548	62.980
9........	2.381	2.405	0.983	2.911	3.930	4.083	4.015	4.215	27.509	25.517	12.988	1.642	67.656
10........	2.603	2.632	1.076	2.992	4.205	4.278	4.253	4.419	28.274	27.303	13.608	1.740	70.925
11........	2.810	2.844	1.162	3.202	4.432	4.521	4.447	4.688	30.259	28.777	14.381	1.819	75.236
12........	0.802	0.804	....	....	....	....	....	....	....	....	....	....	....
13........	2.779	2.812	....	....	....	....	....	....	....	....	....	....	....

TABLE No. 1.— 54-INCH STAVE PIPE.

			Elevations.								Loss of Head.					
			Above Dam. Elev. of Zero, 499.890.		Piez. No. 1. Elev. of Zero, 531.985.		Hook-Gauge. Elev. of Zero, 529.958.		Piez. *A*. Elev. of Zero, 532.445.		Section 1. Length, 232.14 ft.			Section 2. Length, 47.50 ft.		
Number of observation.	Date.	Duration of observation.	Average reading on gauge board.	Elevation of water above dam.	Average reading of Piezometer No. 1.	Elevation of water column.	Average reading of hook-gauge.	Elevation of water column.	Average reading of Piezometer *A*.	Elevation of water column.	Difference in level from dam to Piezometer No. 1.	Loss of head due to friction in pipe.	Observed entry head.	Difference in level from well to Piezometer *A*.	Calculated loss of head in pipe.	Increase of head between well and Piezometer *A*.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
	1901.															
1........	Dec. 14.	10.40 A. M. to 11.08 A. M.	....	....	4.933	536.918	6.126	536.084	....	....	......	......	......	......	......	0.002
2........	Dec. 14.	11.08 A. M. to 12.26 P. M.	....	....	4.932	536.917	6.124	536.082	....	....	......	......	......	......	......	0.002
3........	Dec. 14.	4.20 P. M. to 4.47 P. M.	....	....	4.858	536.843	5.819	535.777	....	....	......	......	......	......	......	0.004
4........	Dec. 15.	9.30 A. M. to 9.56 A. M.	....	....	4.715	536.700	5.383	535.341	....	....	......	......	......	......	......	0.007
5........	Dec. 15.	10.02 A. M. to 10.28 A. M.	....	....	4.716	536.701	5.387	535.345	....	....	......	......	......	......	......	0.007
6........	Dec. 15.	11.00 A. M. to 11.26 A. M.	....	....	4.638	536.623	5.031	534.989	....	....	......	......	......	......	......	0.011
7........	Dec. 15.	11.50 A. M. to 12.16 P. M.	....	....	4.571	536.556	4.698	534.656	....	....	......	......	......	......	......	0.015
8........	Dec. 15.	2.00 P. M. to 2.26 P. M.	....	....	4.515	536.500	4.466	534.424	....	....	......	......	......	......	......	0.018
9........	Dec. 15.	2.40 P. M. to 3.08 P. M.	....	....	4.441	536.426	4.087	534.045	....	....	......	......	......	......	......	0.024
10........	Dec. 15.	3.30 P. M. to 3.56 P. M.	....	....	4.378	536.363	3.802	533.760	....	....	......	......	......	......	......	0.029
11........	Dec. 15.	4.20 P. M. to 4.50 P. M.	....	....	4.284	536.269	3.501	533.459	....	....	......	......	......	......	......	0.034
12........	Dec. 22.	2.46 P. M. to 3.00 P. M.	37.287	....	5.000	536.985	6.211	536.169	3.738	536.183	0.192	0.078	0.114	0.014	0.016	0.002
13........	Dec. 22.	4.03 P. M. to 4.19 P. M.	37.317	....	4.591	536.576	3.817	533.775	1.352	533.797	0.631	0.259	0.372	0.022	0.055	0.033

TABLE No. 2—*(Continued)*.

Number of observation.	Current-Meter Measurements. Velocity, in feet per second. Outer ring, 9.450 sq. ft.	Second ring, 6.493 sq. ft.	Third ring, 3.181 sq. ft.	Center, 0.409 sq. ft.	Discharge, in cubic feet per second. Outer ring.	Second ring.	Third ring.	Center.	Total discharge, in cubic feet per second.	Waste water from settling basin.	Net discharge in 44-in. pipe, in cubic feet per second.	Average velocity, in feet per second.	Loss of head, in feet per 1 000 ft.
(1)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
1	1.634	2.248	2.207	2.183	15.441	14.596	7.020	0.893	37.950	0.515	37.435	3.464	1.067
2	1.706	2.183	2.312	2.264	16.122	14.174	7.354	0.926	38.576	0.515	38.061	3.522	1.134
3	1.755	2.345	2.385	2.296	16.585	15.226	7.587	0.939	40.357	0.517	39.820	3.685	1.191
4	1.852	2.418	2.441	2.425	17.501	15.700	7.765	0.992	41.958	0.517	41.441	3.835	1.262
5	1.892	2.531	2.514	2.507	17.879	16.434	7.997	1.025	43.355	0.516	42.839	3.964	1.330
6	1.893	2.491	2.506	2.505	17.889	16.174	7.972	1.024	43.059	0.516	42.543	3.937	1.331
7	1.941	2.596	2.612	2.555	18.342	16.856	8.309	1.045	44.552	0.513	44.039	4.075	1.404
8	2.321	3.046	3.065	3.042	21.933	19.791	9.750	1.244	52.718	0.512	52.206	4.831	1.888
9	2.224	2.912	2.984	2.895	21.017	18.909	9.492	1.184	50.601	0.512	50.089	4.635	1.757
10	2.211	2.879	2.956	2.879	20.894	18.698	9.403	1.177	50.167	0.512	49.655	4.595	1.757
11	2.126	2.774	2.823	2.782	20.091	18.012	8.980	1.138	48.221	0.511	47.710	4.415	1.613

TABLE No. 2.— 44-Inch Pipe.

			Elevations.						Loss of Head.					
			Hook-gauge. Elev. of "0." 529.958.		Piezometer No. 2. Elev. of "0." 527.174.		Piezometer No. 3. Elev. of "0." 524.776.		Section 4. Length, 150.6 ft.				Section 5. Length, 4041.02 ft.	
Number of observation.	Date.	Duration of observations.	Average of readings.	Elevation of water in well.	Average of readings.	Elevation of water in Piezometer No. 2.	Average of readings.	Elevation of water in Piezometer No. 3.	Difference in level, well to Piezometer No. 2.	Loss of head due to friction in pipe.	Loss of head due to velocity of water.	Loss in valve and basin.	Difference in level. Piezometer No. 2 to Piezometer No 3.	Loss of head per 1 000 ft. of pipe.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
	1901.													
1....	December 19th..	3.54 P. M. to 4.20 P. M....	5.868	535.826	6.163	533.337	4.250	529.026	2.489	0.160	0.186	2.143	4.311	1.067
2....	December 20th..	10.00 A. M. to 10.28 A. M...	5.768	535.726	5.833	533.007	3.650	528.426	2.719	0.171	0.193	2.355	4.581	1.134
3....	December 20th..	11.00 A. M. to 11.28 A. M...	5.722	535.680	5.647	532.821	3.234	528.010	2.859	0.179	0.211	2.469	4.811	1.191
4....	December 20th..	1.44 P. M. to 2.10 P. M....	5.657	535.615	5.447	532.621	2.747	527.523	2.994	0.189	0.228	2.577	5.098	1.262
5....	December 20th..	3.00 P. M. to 3.26 P. M....	5.592	535.550	5.144	532.318	2.166	526.942	3.232	0.200	0.244	2.788	5.376	1.330
6....	December 20th..	3.28 P. M. to 4.38 P. M....	5.587	535.545	5.140	532.314	2.160	526.936	3.231	0.200	0.241	2.790	5.378	1.331
7....	December 21st..	9.28 A. M. to 9.54 A. M....	5.557	535.515	5.117	532.291	1.843	526.619	3.224	0.211	0.258	2.755	5.672	1.404
8....	December 21st..	11.44 A. M. to 12.10 P. M...	5.118	535.076	7.245	534.419	2.015	526.791	0.657	0.283	0.362	0.012	7.628	1.888
9....	December 21st..	2.14 P. M. to 2.40 P. M....	5.234	535.192	7.411	534.585	2.713	527.489	0.607	0.263	0.334	0.010	7.096	1.756
10....	December 21st..	2 42 P. M. to 3.52 P. M....	5.233	535.191	7.409	534.583	2.706	527.482	0.608	0.264	0.328	0.016	7.101	1.757
11....	December 21st..	4.00 P. M. to 4.26 P. M....	5.362	535.320	7.593	534.767	3.417	528.247	0.533	0.244	0.303	0.006	6.520	1.613

The reading of the gauge board above the dam cannot be considered very accurate, as the smallest subdivision was 3 ins. It was taken to show by the observed entry head (Column 14) what error, if any, there might be in the heights of Piezometer No. 1.

	Observed entry head.	Calculated entry head.	Difference.
Observation 12..............	0.114	0.081	0.033
Observation 13..............	0.372	0.341	0.031

This would indicate that the error in the height of Piezometer No. 1 was very slight, if any; probably less than could be observed on the gauge board, as a part of this difference must have been due to resistance at the entrance.

Column 19 shows the sum of Columns 17 and 18. Column 20 shows the quotient of Column 19 divided by the length given in the heading and multiplied by 1 000. Columns 21 to 24, inclusive, give the average velocity in each of the three rings and the center of the opening at the well. Column 25 gives the average velocity through the average section of the pipe, which is always larger than the average velocity at the opening, as the opening where the current-meter measurements were taken is larger than the average section of the pipe.

Columns 26, 27, 28 and 29 show the discharge, in cubic feet per second, of the respective rings of the opening, and Column 30 the total discharge, which is the sum of Columns 26, 27, 28 and 29.

Table No. 2 shows the principal data from the 44-in. pipe experiments. In these experiments there were only two sections of pipe: Section 4, from the well to Piezometer No. 2, and Section 5, from Piezometer No. 2 to Piezometer No. 3.

In Observations 1 to 7, inclusive, the sluice-gate at the entrance to the 44-in. pipe was open only 9 ins., causing the excessive resistance shown in Column 13.

During Observations 8 to 11, inclusive, the sluice-gate was wide open, showing the resistance at the entrance to be very slight, as this also includes the resistance in the basin and in the conduit from the well to the basin.

The current-meter measurements were made at the same place and under the same system as for the 54-in. pipe, except that from the total discharge is taken the waste waterfrom the settling basin.

Columns 27 and 28 give the final results of velocity and loss of head.

Reliability of Results.—In conducting these experiments, the writer has endeavored to eliminate, as far as circumstances would allow, all sources of error, and confine the probable error within the limit that this method of measurement ought to give. He has, in some cases, perhaps, gone to more pains than necessary in order that the reader

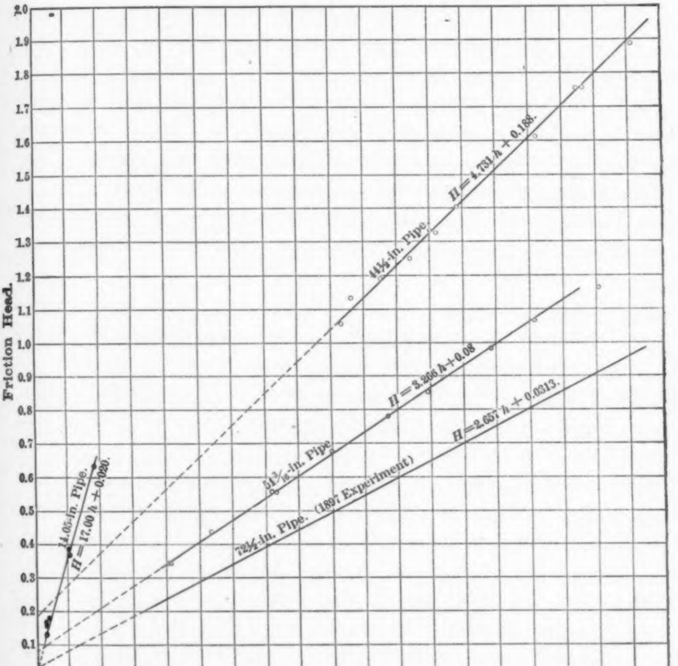


FIG. 10.

may understand clearly the conditions under which the experiments were conducted. If there has been any considerable error for which he has not allowed, it is to be hoped that it will be brought out in the discussion. The current-meter measurements are undoubtedly within 2% of the correct value, and the observations for loss of head are within 1 per cent. The size of the pipe is obtained from so many

separate measurements, 48 in the 54-in. pipe and 100 in the 44-in. pipe, that the probable error in the average size of the pipe must be very small.

Care was taken to guard against the accumulation of air in the connections between the piezometer tubes and the main pipe. That there was no considerable error in this respect is evident from the conformity of the individual observations to one line (see Fig. 10), with the possible exception of Observation 11 on the 54-in. pipe.

All observations of velocity are within 1% of this line, except Observation 11, which is within 2 per cent.

The very slight difference between the calculated and observed velocity head at the entrance of the 54 and 44-in. pipes proves the accuracy of the piezometer heights at Piezometers Nos. 1 and 2. The height in the well is checked by the height at Piezometer *A*.

Comparison with Other Experiments.—As far as the writer has been able to learn, all the available data regarding the loss of head in wood pipes are as follows:

1.—By James D. Schuyler, M. Am. Soc. C. E., on a 30-in. pipe line at Denver, Colo., the details of which are not available. The value of n, in Kutter's formula, is given as 0.0096.

2.—By Mr. Fred B. Gutillius, on the 24-in. pipe of the Butte City Water Company, in which he confirms the use of 0.01 as the value of n, in Kutter's formula.

3.—By Arthur L. Adams, M. Am. Soc. C. E., on an 18-in. pipe for the City of Astoria, Ore. The details of this experiment are given. The results show a value of $n = 0.01$, and $c = 132.9$. Only one observation was taken at a velocity of 3.605 ft. per second.

4.—By Arthur L. Adams, M. Am. Soc. C. E., on a 14-in. pipe for the West Los Angeles Water Company. In all, there were seven observations, the velocity varying from 0.691 to 1.531 ft. per second; value of c, from 99 to 113; value of n, about 0.011.

5.—By Messrs. Marx, Wing and Hoskins, on the 72-in. pipe of the Pioneer Power Plant, at Ogden, Utah. This is by far the most complete and satisfactory set of experiments yet conducted to determine the loss of head in wood pipe. The results have been fully written about and discussed.*

6.—By the writer, on 44-in. pipe, as described in this paper.

7.—By the writer, on 54-in. pipe, as described in this paper.

* *Transactions*, Am. Soc. C. E., Vols. xl and xliv.

In Experiments Nos. 1 and 2, of the foregoing list, no detailed information is available as to the circumstances and conditions under which the experiments were conducted, and in No. 3 no range of velocities was experimented upon.

The only data (so far as the engineering public is aware) which can be taken as criteria for the loss of head in wood pipe are those contained in Experiments Nos. 4, 5, 6 and 7, on 14-in., 44-in., 54-in. and 72-in. pipes, respectively. The range of velocities experimented upon in the 14-in. and 44-in. pipes is not as much as would be desired, and there is a wide gap from the 14-in. to the 44-in. pipe, and from the 54-in. to the 72-in. pipe. It cannot be said, so far as this information goes, that there is any very close conformity in the results, except in the two larger sizes, which agree fairly well; but it is difficult to see wherein they conform with the results on the 44-in. pipe; at least, no general law can be formulated that would seem to apply to all sizes, without the use of a coefficient having arbitrary values for the different sizes of pipe.

Fig. 11, showing the relation between velocity head and friction head, includes only the 1897 experiments made on the 72-in. wood pipe at Ogden. To avoid unnecessary complication, the other two experiments are not given. This experiment was chosen because at that time the age of the pipe was more nearly comparable with the age of the other pipes experimented upon.

It will be seen from Fig. 10 that in all these experiments the velocity head, as compared with the friction head, conforms very closely to a straight line. The evidence is very strong that a formula based upon this hypothesis is correct, as it most nearly fits the experiments so far conducted. Starting with this assumption, the equations for the different sizes of pipe become:

14-In. Pipe:

Let H = Friction head, in feet;

h = Velocity head, in feet per 1 000 ft.;

V = Velocity, in feet per second.

$$H = 17.00\, h + 0.0202$$

$$h = \frac{V^2}{2\,g}$$

$$V^2 = \frac{64.4}{17.00} H - 0.0202$$

$$V = 1.9463 \sqrt{H - 0.0202} \qquad (1)$$

44-In. Pipe:

$$H = 4.731\,h + 0.188$$

$$V = 3.6895\sqrt{H - 0.188} \qquad (2)$$

54-In. Pipe:

$$H = 3.266\,h + 0.08$$

$$V = 4.4405\sqrt{H - 0.08} \qquad (3)$$

72-In. Pipe:

$$H = 2.657\,h + 0.0313$$

$$V = 4.9232\sqrt{H - 0.0313} \qquad (4)$$

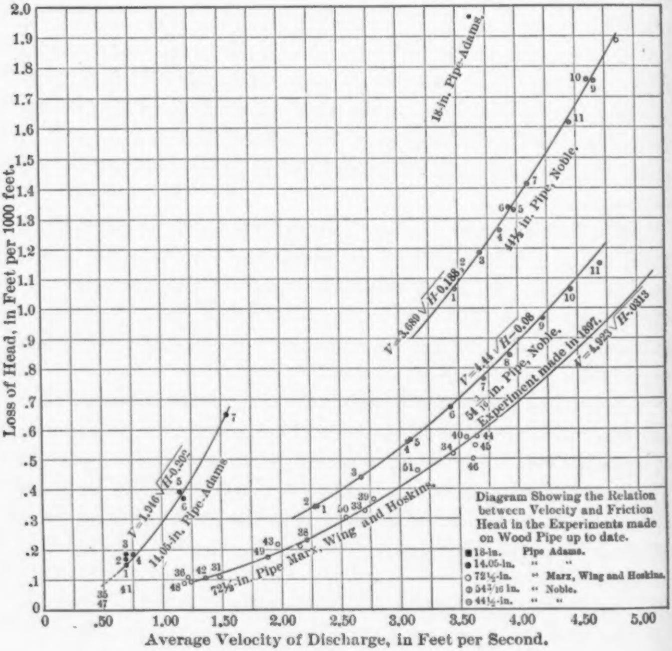


Fig. 11.

On the diagram, Fig. 11, the curves for these several equations (1, 2, 3 and 4), of the relation between V and H, have been plotted. The curves, of course, show the same conformity to the individual observations as in Fig. 10, and will give correct results for the individual sizes of pipe, within the range of the experiments on which they are based, and within the limit of accuracy of these experiments.

Without definite knowledge to the contrary, and until a more complete set of experiments can be conducted, it is safer to use these data as the basis for any formula for wood pipe.

These four equations are applicable to the individual sizes of pipe only, but they fit the experimental data more nearly than any other form of equation possible.

The writer, therefore, would propose the use of a general formula, based on these four formulas, of the following general form:

$$V = e \sqrt{D (H - b)} \qquad (5)$$

For the four sizes of pipe on which there are experimental data, the values of e and b are:

	e.	b.
14-in. pipe	1.799	0.0313
44 "	1.916	0.080
54 "	2.089	0.188
72 "	2.003	0.020

Interpolating for intermediate sizes of pipes, for intervals of 6 ins. in size, these coefficients, in the formula, $V = e \sqrt{D (H - b)}$, are:

	e.	b.
14-in. pipe	1.80	0.031
18 "	1.82	0.041
24 "	1.84	0.073
30 "	1.86	0.114
36 "	1.88	0.145
42 "	1.91	0.176
48 "	1.98	0.146
54 "	2.09	0.080
60 "	2.06	0.063
66 "	2.03	0.047
72 "	2.00	0.031

The values plotted from Formula 5, with the corresponding coefficients, produce the curves shown in Fig. 11. For the intermediate sizes of pipes, the values of e and b are proportional.

The original formulas (1, 2, 3 and 4), are of the form proposed by Mr. Fenkell in his valuable contribution* to the discussion on the second series of experiments on the Ogden pipe line. Mr. Fenkell,

* *Transactions*, Am. Soc. C. E., Vol. xliv, p. 73.

however, proposes that the smaller coefficient, b, is due to errors, and should be dropped. The writer contends that there are no reliable data to support this conclusion, except abstract theory, and that water, like gold, is only where it is found. It is better to swear by the values that we have been able to measure by practical tests than make allowances for errors that may or may not exist.

The foregoing values of e and b are for pipes with smooth interiors, that have not been in use more than one year.

A further factor of safety should be allowed, to cover the increased loss of head when the pipe may be covered with a growth that would retard the flow.

APPENDIX.

Waste Weir.—The 24-in. sluice-gates for draining the basin leak considerably. There is also some leakage through the walls of the basin. This water, with the ground-water around the foundation of the walls, which is all collected into a drain tile surrounding the foundation of the basin, empties into a concrete-lined tunnel. At the mouth of this tunnel a weir, extending the width of the tunnel, was put in for measuring the amount of leakage from the basin. The crest of the weir was 3.04 ft. long, without end contractions; was 18 ins. above the floor of the tunnel, and had a perfectly free discharge. The height was measured from a fixed point, 1 ft. back of the weir, and carefully leveled to the crest of the weir, with a micrometer caliper, reading direct to 0.001 in. The height on the weir varied from 0.1250 to 0.1259 ft. The discharge, according to Bazin's formula, would be from 0.4895 to 0.4955 cu. ft. per second; according to Simpson and Blackwell's formula, 0.5115 to 0.5175 cu. ft. per second; according to Francis' formula, 0.442 to 0.446 cu. ft. per second.

Simpson and Blackwell's formula is probably most nearly correct, and is used to determine the amount of leakage. The leakage in the case of the lowest discharge amounted to 1.38%, and at the highest discharge to 1.07 per cent. Any possible error in the application of this formula would not result in an error of more than 0.15%, which is well within the range of accuracy possible from other causes.

Velocity Head at Well.—After the first eleven observations had been taken on the 54-in. pipe, the writer concluded that, in order to find out whether there was any influence affecting the height of the water in the well which could not be attributed to the friction head, he would conduct two observations, one at the lowest and one at the highest velocity, with an additional piezometer (Piezometer A), located some

distance back of, and away from, any possible influence from the conditions prevailing at the well.

Observations 12 and 13 were conducted for this purpose. The current meter was not used. One observer was located at Piezometer No. 1 and one at Piezometer A, and also one at the hook-gauge in the well. Sixteen readings were taken at each place during each observation, with 1-minute intervals.

This experiment brought out the surprising result that the difference between the heights of water at Piezometer A and the well was less than the amount due to friction in the pipe. This increase of head was 0.002 at the lowest, and 0.033 at the highest velocity. The latter difference was sufficiently large to preclude any possibility of error from leveling, as the increase in difference would not involve the question of leveling.

The cause of this increase was not discovered until the pipe was emptied and the diameter measured, when it was found that the area of the end of the pipe next the well was 16.605 sq. ft., the average area being 16.050 sq. ft.

Assuming that this increase is due to velocity head:

Let V_1 = Velocity in the pipe where it enters the well;
V_2 = Average velocity;
H_1 = Head at the well due to velocity;
H_2 = Head at the average section due to velocity;
H = Velocity head (or gain of head at the well);
A_1 = Area of the pipe at the well;
A_2 = Average area.

Then:

$$H = H_2 - H_1$$

$$H_1 = \frac{V_1^2}{2\,g},\ H_2 = \frac{V_2^2}{2\,g}$$

$$H = \frac{V_2^2}{2\,g} - \frac{V_1^2}{2\,g}$$

$$\frac{V_1}{V_2} = \frac{A_2}{A_1},\ V_1 = \frac{A_2\,V_2}{A_1}$$

$$H = \frac{V_2^2}{2\,g} - \frac{A_2^2\,V_2^2}{A_1^2\,2\,g} = \frac{V_2^2}{2\,g}\left(\frac{A_1^2 - A_2^2}{A_1^2}\right) \qquad (6)$$

Applying this equation, in order to find the calculated value of the gain in head at the well during Observation 13, we have:

$$V_2 = 4.651;$$
$$A_1 = 16.605;$$
$$A_2 = 16.055;$$
$$2\,g = 64.4.$$

From Equation (6), $H = 0.024$.

The observed value of H was 0.033

Taking into account that this difference (0.009 ft.) may be partly due to errors in leveling and partly to error in the area of the pipe at the well, which was measured by two diameters only, the results are sufficiently near to conclude that the effect was due to velocity head.

This effect of velocity head, due to a diverging pipe entering a reservoir, was the subject of a very elaborate series of experiments carried on by the late James B. Francis,* Past-President, Am. Soc. C. E., at Lowell.

He gives a very clear account of the experiments and results, and shows that the maximum effect from a submerged diverging tube of that particular design was an increase of the velocity in the smallest section of the tube of 239% above the theoretical velocity due to the head.

Mr. Francis does not account for this increase. Whether the above formula for head on Venturi, which is the same as given for the Venturi meter, would apply to the results of Francis' experiments, would be interesting to know.

Resistance of Fittings.—As there are, in the entire length of the gravity system, about ninety-five manholes, forty-three blow-offs, twenty-one 3-in. stand-pipes, two 36-in. overflow stand-pipes, thirty-two air-valves and three 36-in. gate-valves, making in all about 196 fittings—not including elbows and T-connections inside the city limits—it was important to find out what influence these fittings have on the total loss of head in the pipe line; it being the original intention to find the loss of head in each kind of pipe, in the gravity system.

For this purpose a differential oil gauge, Fig. 6, was made on much the same plan as described in the paper† by Messrs. Williams, Hubbell and Fenkell, though the writer was not at the time aware of the existence of such a gauge in practical use. The gauge was a rather crude affair, made from such materials as could be obtained in Seattle at the time, and consisted of two glass tubes, $\frac{5}{8}$ in. outside diameter, connected with boiler gauge-glass fittings, so that the tops of the glasses had free communication, and the bottom of each glass could be connected through two pieces of $\frac{1}{2}$-in. hose with the main pipe above and below the fitting, the resistance of which it was desired to obtain. The connection with the pipe was made in the same manner as described for connecting the piezometer tubes.

Both gasoline and coal-oil were used. Gasoline, which was first tried, was found to be objectionable on account of its volatility and its power of absorption. Coal-oil was found to be most serviceable, for the reasons mentioned in the paper by Messrs. Williams, Hubbell and Fenkell.

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* Francis' "Lowell Hydraulic Experiments," pages 222 to 232.

† "Experiments at Detroit, Mich., on the Effect of Curvature upon the Flow of Water in Pipes." *Transactions*, Am. Soc. C. E., Vol. xlvii, p. 1.

The gauge was calibrated by connecting each side with a barrel filled with water. One barrel was provided with a hook-gauge for measuring the height of the surface of the water, and the other barrel with a fixed hook, so that the level of the surface would be maintained. The two barrels were first connected, and the level of the surface in each barrel brought to the level of the fixed hook.

The first readings were then taken from the hook-gauge and differential gauge, with the level in the two barrels the same. The connection between the two barrels was then closed and the level in the barrel containing the hook-gauge was repeatedly lowered, and readings taken until the full range of the scales on the differential gauge was covered.

The results of this calibration made the specific gravity of gasoline 0.720 and coal-oil 0.803. The temperature was about 40° during both experiments.

The result of each experiment is the average of from 15 to 20 readings, with 1-minute intervals.

TABLE No. 3.

	Reading of difference gauge reduced to water head.	Length of pipe.	Friction head in pipe.	Friction head in fitting.
44-in. pipe, Blow-off No. 1........	0.0212	14.20	0.0151	+0.0061
" " " No 2........	0.0149	16.43	0.0175	−0.0026
" " Manhole..............	0.0070	8.36	0.0089	−0.0019
54 in. " Blow-off..............	0.0104	17.00	0.0058	+0.0046

The apparent friction head at these fittings is undoubtedly influenced by the velocity head due to the considerable differences in area which exist in this class of pipe, and would account for the minus quantities in the second and third experiments.

That the differential oil gauge, with proper precautions, is a most delicate and accurate device for measuring differences of pressure, the writer has no doubt. It was some little time before consistent results could be obtained; any irregularity or inaccuracy was readily detected by reversing the hose connections so that the difference of pressure came on opposite sides of the gauge. It was found that even a small amount of air suspended in the water or oil columns, or lodged in the fittings, vitiated the value of the results. This, however, could be eliminated by reversing the connections and taking the average of the two sets of readings. It was also found that globules of water would cling to the sides of the glass in the oil column and affect the results.

The writer's plan for making any future experiments of this nature would be to make the gauge in the form of a continuous, inverted,

glass U-tube, not less than 1 in. in internal diameter, connected at the top with a small glass tube and valve for letting out air and putting in oil. The gauge should be entirely free from any iron or metal fittings where they would come in contact with the oil column, as it is desirable to see the entire column and be sure there is no air or water to reduce or increase the specific gravity of the oil. The connections to the pipe should be free from places where air could lodge, and the main pipe should be tapped at one side of the center to prevent air from entering the connections during the course of the experiment.

In the measurement of velocity head in the Venturi meter, the writer believes that the oil gauge would give much more accurate and reliable results, though he is not aware that it has ever been tried. It is possible that fluctuations might be violent, and interfere with getting a close average. Its effect, however, would be to magnify the conditions prevailing with the mercury gauge, and thus make the average reading more accurate. Where very accurate readings are required with either the piezometer or the differential gauge, better results could be obtained by eliminating the momentary fluctuations of pressure which exist in all pipe lines of any considerable length. This can be done by connecting the pipe leading from the gauge to the main pipe with a reservoir of small capacity; ordinarily a 6-in. stand-pipe would be sufficient where the connection with the main pipe is small. These momentary fluctuations are of no value, as far as the piezometer heights are concerned, and are of constant annoyance to the observer.

This subject also brings to the mind of the writer the possibility of getting much more satisfactory and consistent results in pipe lines by using the differential oil gauge to determine the friction head in the pipe lines, by making connection with a comparatively short length of pipe, say 100 ft. This would eliminate many sources of error. The conditions could be more readily determined, and the loss of head per 100 ft. could be read on the differential gauge direct. This method would seem to be particularly appropriate where the pipes are under considerable pressure, making it possible to dispense with mercury manometers and the complication arising from necessary corrections for temperature. It would be possible to select a length of pipe that would be mostly free from influences outside of the one influence sought. Take the case of the lowest frictional resistance encountered in these experiments, *viz.*, 0.0342 ft. in 100 ft.: The difference gauge using coal-oil would show a difference of 0.1710 in 100 ft., which could be read (after the momentary fluctuations were eliminated) to 0.001 ft., bringing the results within a probable error of 1%, which is better than can be done with ordinary open-tube piezometer readings, where connections are made less than half a mile apart.

Value of Coefficients in Kutter's Formula.—The values for c and n, in Kutter's formula, would seem to conform to the results from the Ogden experiments, but differ widely from Mr. Schuyler's experiment on the 30-in. pipe at Denver, and the experiments by Mr. Adams on 14 and 18-in. pipes.

In Kutter's formula,
$$V = \frac{a + \frac{l}{n} + \frac{m}{s}}{1 + \left(a + \frac{m}{s}\right)\frac{n}{\sqrt{R}}}\sqrt{R\,S}.$$

$$c = \frac{a + \frac{l}{n} + \frac{m}{s}}{1 + \left(a + \frac{m}{s}\right)\frac{n}{\sqrt{R}}}$$

TABLE No. 4.– 54-Inch Pipe.

Observations.	V.	$S \times 1\,000$.	c.	n.
1	2.282	0.342	116.08	0.0130
2	2.276	0.342	115.77	0.0131
3	2.650	0.436	119.38	0.0127
4	3.067	0.558	122.13	0.0125
5	3.045	0.557	121.36	0.0126
6	3.408	0.672	123.66	0.0124
7	3.724	0.783	125.19	0.0123
8	3.924	0.856	126.16	0.0122
9	4.215	0.983	126.46	0.0122
10	4.419	1.076	126.72	0.0122
11	4.688	1.162	129.21	0.0120

TABLE No. 5.—44-Inch Pipe.

Observations.	V.	$S \times 1\,000$.	c.	n.
1	3.464	1.067	110.12	0.0134
2	3.522	1.134	1C8 60	0.0136
3	3.685	1.191	110.88	0.0133
4	3.853	1.262	112.63	0.0132
5	3.964	1.330	112.87	0.0131
6	3.972	1.331	113.05	0.0131
7	4.075	1.494	112.93	0.0131
11	4.415	1.627	113.67	0.0130
10	4.595	1.757	113.83	0.0130
9	4.635	1.757	114.82	0.0129
8	4.831	1.888	115.45	0.0129

The writer can offer no suggestion as to why the value of c should be less and n greater in the 44-in. than in the 54-in. pipe, when, to conform to the results of other experiments, it should be the reverse. The same discrepancy will be noticed in the value e in the formula, $V = e\sqrt{D\,(H - b)}$, proposed by the writer.

The value of n, as in previous experiments, decreases with the velocity, while c increases; whereas the value of e, in the formula, $V = e\sqrt{D(H-b)}$, remains constant for all velocities in the same size of pipe, within the range of the experiments thus far conducted.

The writer is indebted to Mr. Reginald H. Thomson, City Engineer of Seattle, without whose moral and financial assistance the experiments could not have been undertaken ; to Mr. H. W. Scott, Assistant City Engineer, for his assistance as an observer, and for maps and other information regarding the gravity system ; to Mr. E. McCulloh, for his valuable assistance ; to A. H. Fuller, Assoc. Am. Soc. C. E., Professor of Civil Engineering, Washington University, and to Messrs. W. H. Plachy and J. C. Atwood, who assisted in taking the observations.

DISCUSSION.

E. W. Schoder, Jun. Am. Soc. C. E. (by letter).—So much has appeared in recent hydraulic literature* about the use of logarithms and logarithmic cross-section paper in plotting the results of experiments on loss of head in pipes, that it seems remarkable that the author apparently did not avail himself of this most efficient means of examining his results. If the loss of head varies as any power of the velocity, then a logarithmic plotting will yield a straight line, and if the "friction head" varies approximately as the velocity head, then the slope of this line will be approximately 2. Had the author plotted his results in this manner it seems extremely doubtful if he would have proposed the formulas which he did. Mr. Schoder.

The writer presents in Fig. 12 a logarithmic plotting of the data by the author, by Messrs. Marx, Wing and Hoskins, and by Arthur L. Adams, M. Am. Soc. C. E., all referred to in the paper.

It is evident at a glance that the slope of the lines for the 44-in. and the 54-in. pipes is less than 2. As nearly as it can be scaled the slope for both these lines has the same value, 1.73. In other words, the loss of head in the author's experiments varies, not as the square, but as the 1.73ths power of the velocity.

An examination of Fig. 12 will show that the points fit the lines much better than in the author's plotting in Fig. 10. This is to be expected, in view of the fact just stated concerning the actual law of variation. The author has carried on a more consistent set of experiments than he himself was led to believe—a most unusual occurrence.

The data for the 14-in. pipe give the same slope, 1.73, as nearly as can be judged, the number of observations being too few to render an accurate determination possible.

Upon plotting the data of Messrs. Marx, Wing and Hoskins' experiments on the Ogden 72-in. pipe the writer was puzzled by the lack of harmony in the results. Considering first the 1897 experiments on a 2 710-ft. section, the slope of the line best representing the points is almost the same as for the 14-in., 44-in. and 54-in. pipes, but the centers of gravity of the five general groups of points do not even approximate to a straight-line effect. Exercising his best judgment,

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* Osborne Reynolds, F. R. S., in *Transactions*, Royal Society, Vol. 35.
W. E. Foss, Member, Boston Soc. C. E., in *Journal*, Association of Engineering Societies, June, 1894.
Desmond FitzGerald, Past-President, Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 241;
Gardner S. Williams, M. Am. Soc. C. E., in *Journal*, Association of Engineering Societies, Mar., 1901, pp. 170–174;
C. W. Sherman, Assoc. M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. xliv, p. 84;
Messrs. Williams, Hubbell and Fenkell, in *Transactions*, Am. Soc. C. E., Vol. xlvii, p. 180;
Messrs. Saph and Schoder, in *Transactions*, Am. Soc. C. E., Vol. xlvii, p. 314.
Undoubtedly there are many others which have not come to the writer's attention.

Mr. Schoder. the writer has drawn the dashed line in Fig. 12. This line is parallel to the lines for the three smaller pipes. However, if equal weight be given to all the 1897 observations, the slope of the line would be appreciably less than 1.73.

The experiments of 1899 on the long section (22 710 ft.) give a much better line, the slope of which, however, is 1.94! The 1899 experiments on the short section do not give a straight line, but the points, in a general way, check roughly the evidence of the long section just mentioned.

How is this condition of affairs to be explained? The same experimenters on the same pipe line, in 1897, find that the loss of head varies about as the 1.73ths power of the velocity and in 1899 find that the variation is as the 1.94ths power of the velocity! In other words, for the designed capacity of the pipe line, or a velocity of 8¾ ft. per second, the 1899 experiments indicate a loss of head about 40% greater than the 1897 experiments!

Had the pipe deteriorated in the two years? The author states that there was considerable growth in the 54-in. pipe of the Seattle water-supply system as the result of one year's service. There was not enough, however, to make any appreciable difference apparent between the slopes of the lines for the 44-in. and the 54-in. lines. It is quite certain that any considerable increase of roughness of the interior wall of a pipe will cause the loss of head to vary with an increased power of the velocity.

The writer hopes that the question as to whether there was any difference in the interior of the Ogden line at the two times of experimentation, in 1897 and 1899, may be answered, either by careful examination in the future, or another set of experiments. If the latter should be undertaken it is hoped that some additional device for measuring velocity will be used as a check on the Venturi meters, which are placed just beyond curves which in turn follow a breeches pipe or Y.* The result is that the water reaches the meters in a very much disturbed condition, and, as a consequence, the readings of the up-stream piezometer of the meter are far more unreliable than if a long stretch of straight pipe led to it. The higher the velocity, the more unreliable do the indications become, so that at a high velocity, where measurements are generally considered most reliable, they are here most unreliable.

The assumption that the loss of head varies as the velocity head, or as the square of the velocity, is undoubtedly sufficiently accurate for most cases which occur in designing water-supply systems, especially when the range of velocity is within the range obtained in reliable experiments, and when, as is usually the case, the pipe line in question is of cast iron or of riveted steel, and, perhaps, in addition, is a

*See description by Henry Goldmark, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xxxviii., p. 279.

Mr. Schoder.

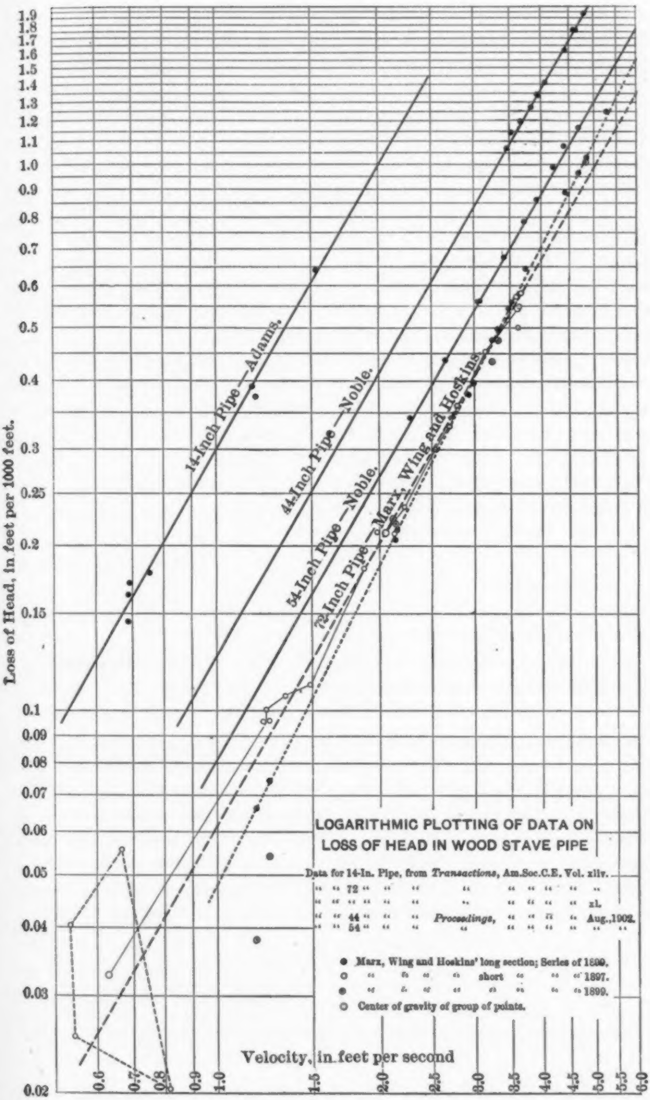

FIG. 12.

Mr. Schoder. feeder for numerous branch lines. In the case of long wood stave pipe lines, however, which are perfectly continuous, as in the case of the Ogden and Seattle lines, it would seem that such an approximation should be replaced by a formula which more nearly expresses the conditions. The only question, as already hinted, is as to the deterioration of the pipe owing to spongy or slimy growths, but, certainly, if it be found that these occur, the engineer will meet the situation by devising effective means for cleaning the pipe and keeping it in the condition for best service.

The experiments plotted in Fig. 12, together with more than 300 experiments* recently performed by A. V. Saph, Assoc. Am. Soc. C. E., and the writer, on fifteen seamless drawn-brass pipes varying in diameter from $\frac{1}{10}$ in. to 2 ins., with every laboratory facility for extreme accuracy, furnish some remarkably harmonious evidence concerning the loss of head in smooth, continuous pipes.

The simplest formula, as well as the most exact one for such pipes, is of the form $H_f = m\ V^n$, where H_f is the loss of head and V is the velocity. The value of m is obtained directly from the logarithmic plotting by noticing the intercept on the line for $V = 1$. The value of n is equal to the slope of the line drawn through the plotted points.

The formulas, of the form $H_f = m\ V^n$, for the four pipes, the data of which are plotted in Fig. 12, are:

14-in.		H_f per 1 000 ft.	$= 0.300\ V^{1.73}$
44 "		" " " "	$= 0.125\ V^{1.73}$
54 "		" " " "	$= 0.0815\ V^{1.73}$
72 "	(1897)	" " " "	$= 0.062\ V^{1.73}$
72 "	(1899)	" " " "	$= 0.048\ V^{1.94}$

To these† may be added the formula given by Mr. Saph and the writer‡ for 2.09-in. seamless drawn-brass pipe:

$$H_f \text{ per } 1\,000 \text{ ft.} = 2.62\ V^{1.759}$$

At the present time the writer does not desire to comment in detail upon these formulas. However, he would again point out the disparity between the formula for the 1899 Ogden series and the formulas for the other five pipes. Considering the variation in the value of m in these formulas, the evidence of the five sets of experiments, for which the exponents of V are practically the same, shows that m varies inversely as the 1.05ths power of the diameter. This power is the slope of a line obtained from a logarithmic plotting of the diam-

* Mr. Saph and the writer hope to be able to present in the near future their experimental data, together with a discussion of the work of others.

† Attention should be called also to the results of Messrs. Williams, Hubbell and Fenkell on a new 12-in. cast-iron pipe, given on pages 180 and 181, *Transactions*, Am. Soc. C. E., Vol. xlvii. This line was laid with great care, and the two 1 000-ft. experimental sections were preceded and followed by long stretches of tangent. The values of m and n, in the equation which fits this pipe, fall with remarkable precision where they would be expected in the logarithmic plottings. The equation as given is

$$H_f \text{ per } 1\,000 \text{ ft.} = 0.3233\ V^{1.779}$$

‡ *Transactions*, Am. Soc. C. E., Vol. xlvii., p. 314.

eter and the values of m. It appears from such a plotting that, for the 44-in. pipe, either the losses of head are about 18% higher or the velocities are about 11% lower than would be expected from the indications for the other four pipes. The author himself has expressed surprise at the non-conformity of the results of the 44-in. pipe with those of the 54-in. and 72-in. pipes. It may be said, however, that the data for the 44-in. pipe are consistent among themselves, and the error, if any, is in the nature of a constant factor. Indeed, the author's results plot up on logarithmic paper in a way which compels admiration and gives strong evidence of careful work. Mr. Schoder.

In conclusion, the writer would say that in laboratory work, even when the utmost care is exercised, apparent inconsistencies are often found, but it is nearly always possible either to find the cause or else by many repeated experiments to obtain convincing evidence that unknown, abnormal causes affected certain results. Unfortunately, the practicing engineer has not the time, nor do conditions generally allow such procedure, in the case of large pipes in use. But the evidence was never stronger than now that there are numerous cases where a hydraulic engineer can estimate closely the performance of a pipe line. On the other hand, the number of cases is legion where only rough approximation is justified. The writer feels that it is in the hope that the dividing line may be more clearly marked that the civil engineer reads with more than passing interest each description of new experiments.

A. V. SAPH, Assoc. Am. Soc. C. E. (by letter).—The writer has been informed by Mr. E. W. Schoder as to the nature of his discussion on this paper, so that there will be no need of attempting a complete discussion of the results of observations on wood stave pipe by plotting them logarithmically and obtaining expressions for the values of m and n in the general formula $H_f = m\ V^n$. For these quantities, Mr. Schoder obtained $n = 1.73$ for most of the experiments, and $m = \frac{k}{D^{1.05}}$. The last result was quite a surprise, as the writer would have predicted that the loss of head should depend on about the 1.30ths power of the diameter instead of on the 1.05ths. One would naturally be led to think that this would be so, as all discussions in regard to cast-iron pipe would have shown this value, as may be seen from the numerous formulas of the form $H_f = m\ V^n$, which have been proposed in late years. Mr. Saph.

The writer, therefore, has plotted the observations on wood pipes and has almost checked Mr. Schoder, as far as the value of n is concerned, but would extend the discussion in regard to the manner in which the values of m depend upon the values of the pipe diameter. In order to obtain this relation, the values of m, as taken from a direct logarithmic plotting of the observed results, are themselves plotted on a logarith-

Mr. Saph. mic sheet as ordinates with the corresponding diameters as abscissas. The slope of the resulting straight line is the required power of D, and its intercept on the axis ($D = 1$) is the value of the constant k. If, on such a plotting, all the values of m are considered, without any attempt at classifying, the mean line would have a slope of 1.05, but, in drawing such a line, it will be noticed that a number of points are situated at quite a distance from it. Assuming, however, that the n for the experiment on 18-in. pipe is 1.74, we get a value of m which, taken in connection with the m for 14-in. pipe, gives a line having a slope of 1.29—more nearly what the writer believes it should be. The experiments by Mr. Noble considered alone would lead to a higher value, but, including with them the experiments by Messrs. Marx, Wing and Hoskins, and taking a mean line, we have here also a slope of about 1.30. The two lines, however, are widely separated, indicating a large difference in the values of the constant k, which is about 0.360 for the smaller pipes and 0.610 for the larger ones. These values apply for losses of head in feet per 1 000 ft. of length, velocities in feet per second, and diameters in feet.

We may mention in addition, the experiment on 24-in. pipe by Mr. D. C. Henny,* M. Am. Soc. C. E., which is usually accredited to Mr. Gutillius.† Mr. Henny says that the experiment is not reliable, on account of the manner of measuring velocities, but it may be introduced here in order to assist in justifying the classification we have made. For a velocity of 1.147 ft. per second the value of c, in $v = c\sqrt{r\ s}$, was 127 (Kutter $n = 0.0103$). Upon determining the corresponding m (under the assumption $n = 1.74$), we find the experiment to be much farther removed from those on the large pipes than were the experiments by Mr. A. L. Adams, and if proper corrections were made it would probably correspond closely with those on smaller pipes. The writer does not believe that these corrections could possibly be large enough to bring it into the same class with the experiments by Mr. Noble and Messrs. Marx, Wing, and Hoskins. Judging by the value of the Kutter n, the experiment by Mr. Schuyler would also fall into the same class with those on smaller pipes.

The writer, therefore, believes that all available experiments on wood pipe fall into two groups, both following the same law as to their dependence on the velocity and the pipe diameter, but differing considerably in the values of the constant, k. Even from a consideration of the values of the Kutter n, this division into groups is made necessary, and the same grouping is reached as by the method used. The cause of the differences found is hard to determine. Messrs. Marx, Wing and Hoskins have said that the differences in the value of the Kutter n can hardly be accounted for through a difference in the

* *Journal*, Association of Engineering Societies. Vol. xxi, 1898.

† *Transactions*, Am. Soc. C. E., Vol. xl, 1898, p. 562, footnote.

roughness of the pipe surface, and the same statement applies to the differences in the values of the constant, k. Curvature suggests itself, then, as a possible cause. The investigations by Messrs. Williams, Hubbell, and Fenkell* show that the long, easy curves cause greater excess losses of head than sharp ones, beyond a limiting radius, which would not occur in wood pipe lines, a fact which has a particular bearing on large wood pipe investigations. The curvature effects, besides depending on the degree of curvature, would also, in a long line of pipe, depend on the ratio of total curved length to total true tangent length. Therefore, in the experiments by Mr. Noble, as well as in all others on wood pipe, it would have been interesting to have had some data in regard to the excess loss due to the curves, in order to account, if possible, for the great difference in experimental results, and also to afford general information in regard to curvature effects. Mr. Saph.

It will also be noticed in connection with the values given for the velocities, that the velocity in the center ring is always less than that in the adjacent ring. Upon plotting these velocities, a very flat curve results, which bears but little resemblance to cycloids or ellipses. This may be due to the enlargement at the well, but it seems hardly probable that it can be wholly accounted for in this way. If it can be, however, it would lead to the conclusion that velocity curves, for both enlargements and contractions, are similar, because similar curves were obtained by John R. Freeman, M. Am. Soc. C. E., for nozzles.†

Reference to the plan of the 54-in. pipe shows that Piezometer A was situated at the point of tangency of a curve, and that there was only a short length of tangent between Piezometer A and the well. Therefore, it would be interesting to know if the individual velocities as measured showed any appreciable effect due to the curve above. The flattened velocity curves might be accounted for in this way, as similar velocity curves are obtained by taking corresponding points from the velocity contour map ‡ presented by Mr. Schoder and the writer in connection with their discussion of the paper by Messrs. Williams, Hubbell and Fenkell. If curvature effects are shown at the well, the averaging of observation to get the velocities in the rings would lead to results somewhat in error. For the same reason, Piezometer A would read low and account for that part of the difference between the water heights at Piezometer A and the well which has not already been accounted for except through possible errors. However, the full natural effect of the curve might not have shown itself on account of the reversal of curvature above.

Mr. Noble's experiments show careful and intelligent work, and have advanced materially our knowledge of the resistances to the flow

*_Transactions_, Am. Soc. C. E., Vol. xlvii, 1902.
† _Transactions_, Am. Soc. C. E., Vol. xxi, 1889.
‡ _Transactions_, Am. Soc. C. E., Vol. xlvii, p. 301.

Mr. Saph. of water in wood stave pipes. There is a need, however, for more experiments of the same kind, especially on diameters between 18 ins. and 44 ins., and, when these are performed, it is hoped that the effects of the curves will be investigated.

Mr. Merriman. MANSFIELD MERRIMAN, M. Am. Soc. C. E. (by letter).—The quantity, b, which is found in the formulas on pages 136–137, does not appear to be due to accidental errors of observation, but probably results from some constant cause. From an inspection of the results given in Fig. 11, it is seen that the highest velocities occurred in the 44-in. pipe, for which the value of b is 0.188, and that the lowest velocities occurred in the 14-in. pipe, for which the value of b is 0.020, while for the other pipes both the velocities and the values of b lie between these limits. This is more clearly shown by Table No. 6.

TABLE No. 6.

Diameter of pipe (inches)	44	54	72	14
Greatest velocity	4.4	4.5	4.7	1.6 ft. per second.
Least velocity	3.5	2.3	1.2	0.7 "
Average velocity	4.1	3.4	2.5	1.0 "
Value of b	0.188	0.080	0.031	0.020

These values of b, at first sight, appear to have little relation to the diameter of the pipe, but it is plain that they increase with the average velocity under which the observations were made.

Roughly, the four values of b deduced by the author are proportional to the squares of the foregoing average velocities in the four sets of observations, as shown in Table No. 7.

TABLE No. 7.

(Average V)^2	16.8	11.6	6.2	1.0
$\frac{1}{100}$ (Average V)^2	0.168	0.116	0.062	0.010
Value of b	0.188	0.080	0.031	0.020

Now, as the square of a velocity is closely proportional to the lost head, it is clear that b, for any set of observations, should be approximately proportional to the average head, H, under which that set was made. Several experiments made under high heads should give a higher value of b than several made under low heads. This is seen to be the case with the four series represented graphically in Fig. 10.

A probable reason has occurred to the writer why the systematic errors represented by b appear in the formulas deduced. It is that the piezometer readings do not give the exact hydraulic gradient which corresponds to the mean velocity of flow in the pipe. The piezometers measure the pressures at the circumference of the pipe where losses

occur by both sliding and eddying friction. These frictional losses are probably greater than those which prevail for a filament where the velocity is equal to the mean velocity. If so, the hydraulic gradient obtained by taking differences of piezometer readings has a steeper slope than that corresponding to the mean velocity of flow. Hence the losses of head per thousand feet, represented by H, are too great, and it has been shown that they are greater for large friction heads than for small ones. The quantity, b, from this point of view, is a correction to be applied to the observed friction head, H, in order to give the true friction head, $H - b$, which corresponds to the mean velocity, V. Mr. Merriman.

The writer puts forth this hypothesis as a tentative one, only, trusting that it may lead to further discussion. He is not fully convinced that the hypothesis gives the true reason why b appears in the formulas, but, that the values of b vary with the average velocity head in the different sets of observations, appears to be fully established by the data of the paper.

Using the author's notation, the formula for the mean velocity, V, in a pipe of diameter D, due to a loss of head of H feet per thousand, is

$$V = e\sqrt{D(H - b)}.$$

Now, as b^2 is small compared with H, an approximate value of V^2 is $e^2\,D\,H$, and as it has been shown that b, for these wood pipes, is closely proportional to V^2, an algebraic expression for it is,

$$b = \text{constant} \times e^2\,D\,H.$$

The quantity b, therefore, turns out to be proportional, both to the friction head, as deduced from the piezometer readings, and to the diameter of the pipe.

The values of b computed from this expression, taking the constant as 0.009, are shown in Table No. 8. A comparison of these with the values deduced by the author shows an agreement which is perhaps sufficiently close to give some support to the hypothesis advanced by the writer.

TABLE No. 8.

Nominal diameter (inches)	14	44	54	72
Diameter D	1.17	3.71	4.52	6.04 ft.
Value of e	1.80	1.92	2.09	2.00
Average H	0.30	1.43	0.70	0.31 ft.
$0.009\ e^2\ DH = b$	0.010	0.176	0.114	0.067
Author's b	0.020	0.188	0.080	0.031

Substituting the algebraic expression for b in the foregoing velocity formula, it becomes,

$$V = e\sqrt{D\,(H - 0.009\,e^2\,DH)},$$

in which the quantity, $0.009\,e^2\,D\,H$, may be regarded as a correction to be subtracted from the observed friction head, H, in order to reduce it to the friction head which corresponds with the mean velocity, V.

Mr. Merriman. The single observation made by Mr. A. L. Adams on an 18-in. wood pipe furnishes the data for computing e by the help of the foregoing formula, as all the other quantities are known. Thus, from $V = 3.63$ ft. per second, $H = 1.97$ ft. per thousand, and $D = 1.50$ ft., there is found $e = 2.18$, and then $0.009\ e^2\ D\ H = 0.128$, whereas,

$$V = 2.18 \sqrt{D(H - 0.128)} = 2.67 \sqrt{H - 0.128}$$

is the formula for this wood pipe under the given data. Had the observation been taken under a smaller head, the value of b would have been smaller. The interpolated values for this case given by the author on page 137, are $e = 1.82$ and $b = 0.041$. The above discussion indicates, however, that interpolation in this case is not fully justified, for, although 18 ins. lies between 14 and 44 ins., the mean velocity and the friction head for the 18-in. pipe are outside of the limits of those observed on the other two pipes, as is seen by Fig. 11.

The values of the coefficient, e, deduced by the author, generally increase with the diameter of the pipe. This is in agreement with the law usually found to hold for pipes and conduits. How e varies with H, however, is not apparent from the data. According to Kutter's formula, e should increase with H when the hydraulic radius of the pipe is less than one meter, and this is the case with all these wood pipes. According to Bazin's formula, e should be independent of H. It is clear that the influence of H upon e will be much less than that of D. The writer has made several attempts to deduce a formula for e in terms of H and D, from the author's data, but they have not been sufficiently successful to record here. Probably the different degrees of roughness of the pipes obscure the influence of the different heads. For the present and immediate future the values of e deduced by the author in his interesting and valuable paper must be used for wood pipes, but the present discussion indicates that the probable value of the quantity, b, may advantageously be computed from

$$b = 0.009\ e^2\ D\ H.$$

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—The speaker believes this paper to be a useful contribution to knowledge, but regrets that the author has presented a new formula which he says fits his experiments very well, because the more formulas of this kind we get, the greater the confusion becomes in their application. If the application were always limited to exactly the conditions which the author had before him, the formula would be good, without question. But, when getting into textbooks, and when applied in general practice, they are often disconnected from the original data from which the formulas are developed, and unless we are within the original range of experiments their use becomes dangerous.

The speaker was once very forcibly struck with the importance of this fact by a formula suggested for pile-driving by one of our most prominent engineers, and found in at least one textbook. When apply-

ing it, it gave negative results in connection with the case in hand, which was quite an ordinary one; that is, the harder the pile was pounded, the less its firmness—according to the formula. When any one produces a new formula, the fact that it applies only to his own experiments or investigations should be emphasized, and the limits should be given as a part of the formula, just as limits are given when the integral calculus is used. Mr. Hering.

Gardner S. Williams, M. Am. Soc. C. E.—To judge from the limited number of discussions presented, it seems that the merits of the paper under consideration have been but lightly appreciated. Mr. Williams.

Anyone who studies the literature upon the flow of water in pipes will be impressed first of all by the wide gaps existing between the sizes used in the several experiments, the data of which are available, and then by the number of experiments that must be rejected from a classification intended to include only the thoroughly reliable.

On wooden stave pipe, naturally, from its comparatively recent introduction, only a very limited number of experiments has been thus far conducted, and of these, the most extensive series appears to have been influenced by conditions beyond the control or conception of the observers, to such an extent as to render the results obtained, if not erratic, at least so variable and unconformable to any apparent law that their critical study leaves one in very great doubt as to their general applicability. It is doubtful if anyone, outside of the experimenters themselves, has spent more study than the speaker upon those experiments, and certainly no one has striven more faithfully to glean from them some measure of information generally applicable to the flow of water in pipes, but thus far the Ogden Experiments* have stood at the extreme end of a series, far removed from all others, and presenting results widely different from everything approaching them; and the only way to explain them has been to say that somewhere between the conditions of ordinary-sized pipe and those of a 6-ft. conduit the laws of flow are subject to a marked change.

The experiments described in Mr. Noble's paper bring the data much nearer to the Ogden size, and it is important to notice that there is in them no indication of an approach to the Ogden eccentricities, but they conform to the indications of other standard experiments and show that the carrying capacity of wooden stave pipe is considerably higher than the Ogden results would lead one to expect.

But, aside from this, one of the most interesting features of the investigation is the light it throws upon the inapplicability of the long-honored law that loss of head varies as the square of the velocity. Ever since the days of Du Buat, Eytelwein and Prony, many investigators have, from time to time, pointed out that this law is not strictly

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 471, and Vol. xliv, p. 34.

Mr. Williams. applicable to the flow of water in pipes. In 1808, Dr. Thomas Young suggested, in an address to the Royal Society,* that this exponent was more nearly 1.8, although he inclined decidedly to the view that there were two functions of the velocity involved, one varying as the square and one as the first power. Later, in 1855,† Thomas Hawksley made an extended exposition of the same view, and in 1873 Dr. Lampe‡ presented his experiments on the Danzig pipe line, with his well-known formula, $V = 203.3\, r^{0.694}\, s^{0.555}$. During the last ten years numerous investigators have suggested various exponents other than 2 for V, and some time ago the speaker himself presented, to the Detroit Engineering Society, a discussion of the past experiments on small pipes bearing upon this question.§ Starting with the formula $H_f = mV^n$, which is about the simplest expression for the flow of water, m and n being constants for any given pipe line, he attempted to discover what effect, if any, there was upon the exponent n due to changes of diameter, alignment and character of interior surface. From an investigation of more than eighty different series of experiments by thirteen different observers, certain conclusions were drawn regarding the variation of n which seemed clearly established by these data, and the principal ones were:

That n increases as the diameter increases, from about 1 in capillary tubes to about 2 in large pipes, being from 1.80 to 2 in those ordinarily used by the engineer.

That n increases as roughness increases.

That n decreases as curvature increases.

That n is different for different materials, being lowest for tin and brass.

This investigation having shown the desirability of a series of experiments upon pipes of the same material and different diameters under similar conditions, such a series was begun at the Hydraulic Laboratory of Cornell University by A. V. Saph, Assoc. Am. Soc. C. E., and E. W. Schoder, Jun. Am. Soc. C. E., and, to date, observations have been made upon pipes of seamless brass tubing, ranging from $\frac{1}{10}$ in. to $2\frac{1}{10}$ ins. diameter. As stated in the discussions by those gentlemen, these experiments show n to be practically constant for all of the series and approximately 1.75.

The experiments of the author, when discussed logarithmically by the formula $H_f = mV^n$, also give for the value of n about 1.75, a result which, taken with that of the Cornell Experiments, seems to establish this value pretty clearly as the correct one for smooth pipe, and proves in error the speaker's conclusions, previously mentioned, that n changes with the diameter, as it shows that as long as the pipe

* *Philosophical Transactions*, 1808.

† *Minutes of Proceedings*, Institution of Civil Engineers, 1855.

‡ *Der Civilingenieur*, Vol. xix, 1873.

§ *Journal*, Association of Engineering Societies, March, 1901, p. 169.

remains smooth n does not change. When, therefore, starting with a diameter of $\frac{1}{10}$ in. and going to a diameter of 54 ins., including the Adams experiments on 14-in. stave pipe, the exponent is found to remain constant at about 1.75, what is to be said for an exponent of 196, when a 72-in. pipe is reached? Mr. Williams.

Some two years ago a series of experiments was made at Detroit* upon a very perfect line of 12-in. pipe. There was a tangent of about 3 200 ft. that was as straight and true to grade as pipe can be laid, and, when this formula was applied to the experiments upon it, the exponent of n was found to be 1.78. The cast-iron pipe experimented upon by Darcy gave $n = 1.93$ to 1.97, and it will be recalled that, in the experiments upon the Rosemary Syphon†, Desmond FitzGerald, M. Am. Soc. C. E., found n to be 1.91 for the cleaned 48-in. pipe, and 2.03 while the pipe was roughened by tubercles.

It is evident that in measuring lengths in diameters there are more joints, and, hence, greater roughness, in large than in small cast-iron pipe, and from the fact that the ordinary experiments upon it give n a value of about 1.90, the effect of careful laying is shown in the Detroit pipe, and the cause of the speaker's misconception of the true law is at once apparent, for, in drawing the conclusions above stated, the most complete series used was of cast-iron pipe, and in this, for the above reason, the exponent does increase with the diameter, though, as it now appears, not by reason of the diameter, but by reason of the joints and consequent roughening of the walls.

The experiments of the author show a high degree of consistency when examined carefully, and indicate very good experimental work. If there were an error in the rating of the meter the error is carried consistently through each series, and if there were errors of levels or observations they were consistent errors, for there are no erratic results to be accounted for, and it seems unlikely that the relations indicated between H and V, so far at least as n is concerned, are misleading by more than a very small percentage. It is unfortunate, however, that the author himself did not apply the formula $H_f = m\ V^n$ to his results, for, had he done so, he would not have presented the complicated expressions on pages 135 and 136, or the table on page 137 in which the value of his own experiments is to a considerable extent masked by an attempt to make them conform to the less reliable results obtained at Ogden. The presence of the intercept or subtractive term in these formulas indicates that the exponent of V is not 2, as has been shown by the speaker in the discussion already referred to.‡

The speaker would call attention to Table No. 3, and offer a word of caution as to the applicability of the results therein presented. While these data undoubtedly give a correct representation of the

* *Transactions*, Am. Soc. C. E., Vol. xlvii, p. 159.

† *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 258.

‡ *Journal*, Association of Engineering Societies, March, 1901, p. 170.

Mr. Williams. losses of head observed from one side to another of the specials in question, those results come far short of indicating correctly the resistances actually caused by them. The actual losses may be either greater or less than shown by the figures given, for the reason that the disturbances of flow thus generated extend with corresponding losses of head for many diameters beyond the special causing them; and piezometric indications within the region of disturbance are not comparable with those taken before disturbance has begun. The only method thus far known to the speaker of determining correctly such resistances is by comparing the losses in sections of straight pipe without such specials with those of other sections containing them. The speaker and his associates in the Detroit Experiments were led to the same attempt at the determination of these losses as the author has made, but they found the results thus obtained anomalous, and the experiments described by Messrs. Saph and Schoder* threw such additional light upon the matter that in their final discussion of the Detroit Experiments the authors went to considerable pains to re-compute the various losses of head, according to the plan proposed above, the results being presented in tabular form,† and showing quite decided differences from those obtained by direct observation.

Mr. Noble THERON A. NOBLE, M. Am. Soc. C. E. (by letter).—In the experiments on "The Flow of Water in Wood Pipes" the main effort was expended in eliminating sources of error and in making the experiments as complete as the time and facilities available would permit. That sufficient study was not given to the results was due to lack of time. It is rather a serious loss of time to a practicing engineer to undertake such an extended series of experiments.

The writer believes, with Mr. Hering, that an excess of formulas, like an excess of advice, is apt to lead to wrong conclusions. It is very bad practice, however, to cling to old methods when they have been found to be cumbersome and inaccurate. If the innumerable experiments are to teach us nothing of the laws of motion of water in pipes, we are losing their main value. By a careful study of these experiments the writer believes that a reliable formula is to be had for each particular kind of pipe, within certain ranges of velocity and diameter.

In the case of wood pipes, the number of experiments available is not all that could be desired to determine the relation between velocity, friction head and diameter, but a formula can be determined which will fit three out of four of the experiments, and determine the values of V, H_f or D, well within the limits of ordinary practice. This formula applies to wood pipes from 14 to 72 ins. in diameter, and friction heads from 0.2 ft. to 2 ft. per 1 000 ft. This is certainly a step in advance.

* *Transactions*, Am. Soc. C. E., Vol. xlvii, p. 317.

† *Transactions*, Am. Soc. C. E., Vol. xlvii, p. 360.

A similar study of the experiments on cast-iron and other pipes, the writer believes, would give similar results, with a somewhat different relation between D and the constant coefficient e'. Mr. Noble.

Kutter's formula, which was particularly designed for the flow in open channels, is wholly inadequate for use in calculating the flow in circular pipes, for the following reasons:

The coefficients, c or n, are found to change very radically with both the diameter and velocity, thus making it necessary to consult tabulated values of these constants for different diameters and velocities in order to get results at all satisfactory. It involves a wholly unnecessary determination of the mean hydraulic radius, R, since R is the diameter times a constant multiplier, which could just as well appear in the coefficient.

In a paper* by George H. Fenkell, Jun. Am. Soc. C. E., he shows by diagrams the results of all the more reliable experiments conducted on various kinds of pipes showing any considerable range of velocities. All these observations, in each experiment, he has plotted on cross-section paper, showing the relation between velocity head and friction head. In all these diagrams this relation conforms very closely to a straight line for the observations on any single size or kind of pipe. The deviation of any one of these observations is probably within the limit of error due to taking observations or making calculations. The same is true of the two experiments conducted by the writer. This relation between H_v and H_f gives a reliable basis for determining the relation between D, in the formula, and the value of the constant.

Starting with Mr. Fenkell's general formula for the relation between H_v and H_f:

$$H_f = a\,H_v \pm b \qquad (7)$$

In which H_f = Friction head, in feet per 1 000 ft.;

H_v = Velocity head $= \dfrac{V^2}{2g}$;

a = A constant, which remains constant for different velocities, but is different for different values of the diameter, D;

b = A constant of small value, which Mr. Fenkell attributes to some error, but which is more probably due to the causes mentioned by Professor Merriman.

Substituting the value of V,

$$V^2 = \frac{2g}{a}\,(H_f \pm b)$$

$$V = \sqrt{\frac{2g}{a}}\,\sqrt{H_f \pm b} \qquad (8)$$

* *Journal*, Association of Engineering Societies, March, 1902.

Mr. Noble.

The value of $\sqrt{\frac{2g}{a}}$, as shown in Formulas 1, 2, 3 and 4, is as follows:

Size of pipe, in feet.	$\sqrt{\frac{2g}{a}}$
1.171	1.9463
3.708	3.6895
4.5156	4.4405
6.042	4.9232

For wood pipe of any given size, $\sqrt{\frac{2g}{a}}$ is constant, and, therefore, is independent of the value of H_f, or V, but increases by some unknown relation with D.

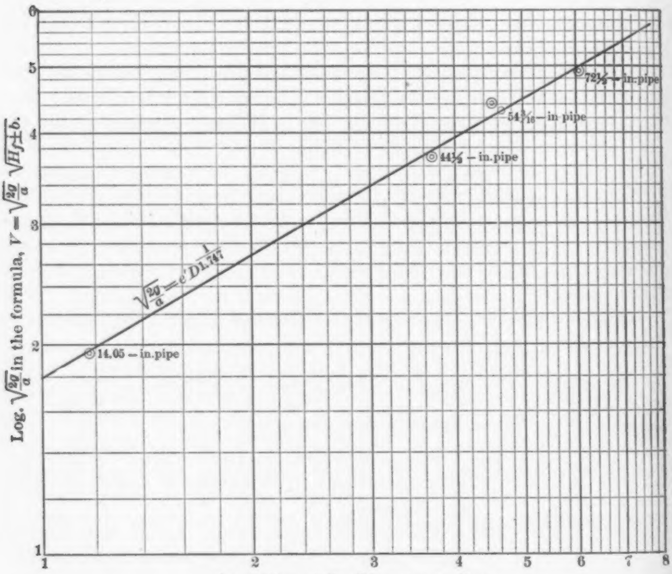

FIG. 13.

It will be seen (page 137) that the assumption that $\sqrt{\frac{2g}{a}}$ varies as $\sqrt{D}$, is not correct, as the resulting values of e are not constant for different sizes of pipes.

Is there any other power of D, which, multiplied by a constant, will give the correct values of $\sqrt{\frac{2g}{a}}$ for the different sizes of wood

Mr. Noble.

pipes experimented upon? If so, the values of $\sqrt{\frac{2g}{a}}$ and D, plotted on logarithmic paper, would appear in one straight line. In the diagram, Fig. 13, these quantities have been thus plotted.

As will be seen, in three out of four of these experiments, the plotted values of $\sqrt{\frac{2g}{a}}$ and $\sqrt{D}$ conform to a straight line very closely. Drawing the line through the points for the 14-in. pipe, and the center of gravity for the other three sizes, the value of the exponent of D becomes $\frac{1}{1.747}$, and Formula (8) becomes

$$V = e' D^{\frac{1}{1.747}} \sqrt{H_f \pm b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

To determine the value of e':

$$V = e' D^{\frac{1}{1.747}} \sqrt{H_f \pm b} = \sqrt{\frac{2g}{a}} \sqrt{H_f \pm b}$$

$$e' D^{\frac{1}{1.747}} = \sqrt{\frac{2g}{a}}$$

$$e' = \frac{\sqrt{\frac{2g}{a}}}{D^{\frac{1}{1.747}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

The values of e, as used in Formula (5), and e' in Formula (9), for pipes of different sizes, are:

Diameter of pipe, in feet.	e	e'	Deviation of e' from 1.76
1.171	1.799	1.777	+ 1 per cent.
3.708	1.916	1.742	— 1 "
4.5156	2.089	1.873	+ 6.4 "
6.042	2.003	1.758	— 0.01 "

With the exception of the 54-in. pipe, the value of e' does not vary more than would be expected, from errors in taking observations, etc. Better results could not be obtained without more perfect similarity of conditions and more accurate methods of measurement, and these are quite close enough for general engineering practice.

The writer feels satisfied that there is some error either in the measurements or the conditions in the 54-in. pipe experiments, and, some time in the future, hopes to conduct a series of observations on that portion of the pipe which is comparatively free from distortions, and by methods which will give more accurate measurements of piezometer heights.

In Fig. 14 the curves for Formula (9), with $e' = 1.76$, are drawn for the four sizes of pipe experimented upon. The greatest variation from this curve is in the 54-in. pipe. In the case of the other three

Mr. Noble. the observations are close enough for ordinary engineering practice, thus proving that the formula could be used (within the limits hereinafter mentioned), and be within a probable error of not more than 2 per cent.

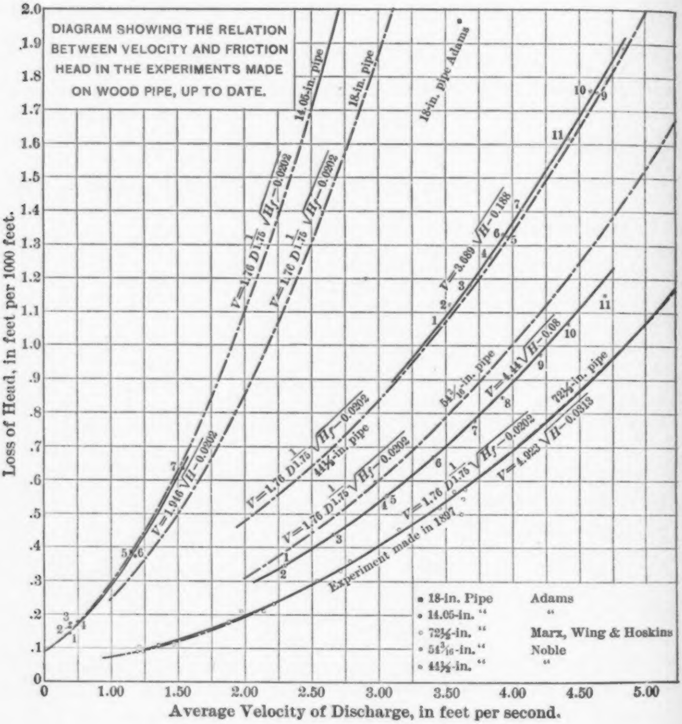

FIG. 14.

In Mr. Schoder's discussion, in plotting the velocity and friction head on logarithmic paper, the observations at the higher velocities are bunched together in such a manner that it is not possible to determine the slope of the line with much accuracy. In the writer's opinion, Mr. Fenkell's diagrams prove, beyond question, that H_f varies as the square of the velocity.

This would seem to be a natural conclusion, from the laws of gravity. Whatever other influence there is on the friction head would be due to the diameter, or to the nature of the surface of contact. The effect of friction in the pipe is to reduce its effective diameter, which

is accounted for in the value of the exponent $\frac{1}{1.747}$. The difference between Mr. Schoder's conclusion, as to the exponent of H_f (1.73), and the writer's is perhaps due to the fact that the writer has accounted for this variation in the diameter rather than in the velocity. Mr. Noble.

In answer to Mr. Saph's contention that the height of Piezometer A is probably influenced by the curves just preceding: Any influence of this nature would be felt in the well and at Piezometer A equally. There are no means of detecting any influence of this nature without taking observations on straight and curved portions of the pipe simultaneously. It is possible that this influence may account for the lack of conformity of the 54-in. pipe experiments with the others, though the writer does not believe that such an effect exists, as the effect of curves should be to increase rather than decrease the friction head; and it seems hardly possible that long easy curves could increase the velocity 6 per cent.

As will be seen in Fig. 14, the single observation of the flow in 18-in. pipe shows a very serious difference from the results obtained for other pipes, showing again higher velocities.

As the result of careful study of these experiments, the writer feels justified in drawing the following conclusions :

1.—That within the range of these experiments, from 14 in. to 72-in. pipes, and between friction heads of 0.2 ft. and 2 ft. per 1 000 ft., the friction head, H_f, varies as the square of the velocity.

2.—That the velocity varies as some odd exponent of the diameter. In three out of four experiments this exponent is very nearly $\frac{1}{1.75}$.

3.—That it would require a series of experiments on pipes of different diameters, and velocities covering a wide range, made under identical conditions, to determine this exponent accurately.

4.—After this exponent has been determined, the flow in perfectly straight and smooth pipes follows a definite law. That the general expression for this law, as determined by the writer, is

$$V = e' D^{\frac{1}{n}} \sqrt{H_f \pm b}.$$

5.—That the formula which conforms very closely to three out of four of the experiments is

$$V = 1.76\, D^{\frac{1}{1.75}} \sqrt{H_f \pm b}.$$

6.—That the use of this formula will probably give results within 2% of true values.

It is probable that, by a set of very accurate experiments made at low velocities, the value of b may be zero, and that the constant e' and the exponent of D may be changed somewhat to make the formula conform to the experiments.

Mr. Noble. The amount of thought and attention that Mr. Williams has given the subject of the flow of water in pipes makes his discussion a valuable addition to the paper. The writer has had neither the time nor the facilities to go as deeply into the subject, from the standpoint of what has been done with other kinds of pipe, and, in the paper and discussion, has confined his attention exclusively to the kind of pipe and within the range of sizes which these experiments covered. His object has been two-fold: To furnish the data in its original form, with all steps in the calculations, so that it could be fully discussed and utilized; and to put the results of the only experiments conducted on wood pipe in such form that the practicing engineer who has use for data of this nature could calculate the flow for any intermediate size or slope and feel satisfied that the results of his calculations would conform to the reliable experiments extant.

Whether the velocity varies as an odd exponent of the diameter or of the friction head is a question as to the manner in which the formula is derived. That the velocity should vary as the $\frac{1}{2}$ power of the head, and that the diameter should vary as some odd exponent with the velocity, would seem to the writer to be most conformable.

What Mr. Williams says in regard to the experiments on loss of head in specials is undoubtedly true. The original data were given to demonstrate that there could not have been in these specials any resistance that would affect materially the total friction head in the lengths of pipe experimented upon, the object being to eliminate a probable source of error.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 939.

THE FOOTBRIDGE FOR BUILDING THE CABLES OF THE NEW EAST RIVER BRIDGE.*

By Isaac Harby, Jun. Am. Soc. C. E.

With Discussion by W. Hildenbrand, M. Am. Soc. C. E.

The erection of one suspension bridge from which to build the cables of another has introduced new ideas and instituted a new procedure in the methods of constructing bridges of this type. The cables of the New East River Bridge No. 2, also designated as "The Williamsburg Bridge," are novel in other respects. They are the largest suspension bridge cables ever attempted, and are suspended from towers taller than those in use in any other bridge. These towers are also distinguished by being the first large ones of steel to be used for this purpose.

It is not unnatural, then, that new methods should characterize the work of building such cables.

The engineer in charge of the contract for constructing these cables, Wilhelm Hildenbrand, M. Am. Soc. C. E., whose assistant the writer is, has invented a method of doing better work of this class in a shorter time than has previously been accomplished on any other similar undertaking.

Before entering into the main subject of this paper, a more comprehensive idea of the situation may be given by a brief description of the main cables themselves, the erection of which was the reason for the existence of the footbridge. The conditions encountered at the beginning of the undertaking will also be stated briefly.

* Presented at the meeting of November 5th, 1902.

There are to be four main cables, each composed of 37 strands of 208, No. 6, steel wires, laid straight; all of which are finally to be squeezed into one cylindrical cable measuring about 19 ins. in diameter.

The distance between the centers of the towers is 1 600 ft. The cables are to be passed over the tops of these, where each one is to rest in a saddle mounted on a roller bed which permits the saddle to move 3 ft. in the direction of the cable.

The weight of the main span of the bridge is to be carried directly by the cables, but the parts of the bridge between the towers and the anchorages are to be otherwise supported. Thus the back stays, or shore spans, of the cables will have no load suspended from them. In order to equalize the tensions in the shore spans and the main span of the cables during their construction, the saddles are moved back on their roller beds 3 ft. toward the anchorages and held there. This position has been determined by calculation as the proper one to balance the forces caused by the tension in the unloaded cable. The saddles will move forward gradually during the erection of the superstructure of the main span.

The elevation of the center of the cable where it will rest in the saddle is to be 332.708 ft. above mean high water, and the elevation at its lowest point in the main span is to be 161.027 ft. The cables will be anchored 617 ft. (measured horizontally) from the center of each tower, at an elevation of 76.627 ft. The end of each strand will be looped around a cast-steel shoe, held by a pin passed through the end of a chain of eye-bars built in a masonry anchorage.

During the spinning of the wires it is necessary to support each strand on sheaves a little above the position it will finally occupy in the saddle, and also to hold each end of the strand a little back of its final position in the anchorage. The combined result of this condition will cause the center of the main span of each strand to hang about 14 ft. higher while it is being made than it will after it is lowered into its final position.

Reasons for Constructing a Footbridge.

On previous work of this kind a so-called footbridge was used, but it consisted merely of a single path from anchorage to anchorage, used only as a means of passage, and not intended as a working platform.

FIG. 1.—NEW EAST RIVER BRIDGE NO. 2: TOWERS AND END SPANS READY FOR CABLES.

FIG. 2.—REELS OF WIRE ROPE ON FLOAT BEING TOWED ACROSS RIVER AND LAID ON THE BOTTOM.

In the case of the New York and Brooklyn Bridge, this footwalk was suspended on two $2\frac{5}{8}$-in. wire ropes. When the Cincinnati and Covington Bridge (the second largest suspension bridge now in use) was recently rebuilt by Mr. Hildenbrand, by the addition of extra cables above the old ones, the footbridge there in use was laid on top of one of the old cables.

The wrapping of the strands and of the finished cable has previously been done from travelers suspended from the cable itself. The regulation or adjustment of the wires has been accomplished by the assistance of cradles, suspended at several points along the line, from which men could reach the wires during regulation.

These methods left much room for improvement, and it became quite evident that if some sort of a working platform could be devised, extending the full length of the cables to be built, and from which the cables could be reached at all points and at all times, a much better cable could be built in a much shorter time.

Thus the demand for such a working platform called into active service the inventive mind of the engineer in charge of the execution of the contract, and he has designed an original as well as an ingenious method of building large cables.

Description of the Footbridge.

This footbridge is more than an ordinary footwalk from tower to tower. In the main or river span it is a double-deck bridge, and consists of eight continuous footways, four above and four below. The four upper footwalks are about 4 ft. below the line which will be occupied by the strands of the main cable during the time of spinning. The four lower walks are just below the line to be occupied by the cable when the strands have been placed in their permanent position.

These different footwalks are connected by cross-bridges at numerous points, so that an easy communication is afforded from all parts of the structure.

The two land spans are placed directly below the line of the cables, and have only a single deck of four walks.

The whole structure is supported by sixteen wire ropes assembled into four groups of three ropes each, with a single rope suspended above each group. These ropes are stretched from anchorage to anchorage, and passed over the tops of the towers in saddles especially provided for them.

The three ropes composing a cable are clamped together with iron bands every 5 ft., so as to keep the three together throughout their entire length. They are $2\frac{1}{4}$ ins. in diameter, and are made of seven strands of galvanized steel wires, twisted together. The ultimate strength of each is 208 tons, and the weight 9 lbs. per foot. Each rope was made long enough to extend the entire distance from anchorage to anchorage, about 3 020 ft.

The ends of these cables were made fast to a 30-in. box-girder spanning an opening in the face of an inner wall of the anchorage, the girder being set at such an inclination that the end of each wire rope could pass through it parallel to its web. The ropes thus passed through are secured by means of button sockets which bear against the back of the girder.

In order to permit of some adjustment in the lengths of the different ropes forming the cables, and also for convenience in erecting, each is pieced, about 100 ft. from each end, and reunited by means of sockets with screw rods passing through them. These rods are $3\frac{1}{4}$ ins. in diameter, $6\frac{1}{2}$ ft. long, and are threaded at each end. They permit of an adjustment of 4 ft.

The ends of the four ropes are spaced along the anchorage girder about 1 ft. apart, and the ropes converge to a point on the front wall of the anchorage where they pass over a cast-iron saddle. Between this saddle and the saddle on the main tower they are clamped into one cable, and form the support for the land span of the footbridge.

The clamps are spaced every 5 ft., measured horizontally, and from each hangs a suspender rod of $\frac{3}{4}$-in. round iron and an inclined 1-in. suspender reaching sideways. Between each pair of cables is a $\frac{5}{8}$-in. tie-rod. The lower ends of the suspenders pass through 3 x 8-in. yellow pine beams, upon which the 2 x 6-in. flooring is laid. As the slope of the land span becomes steeper as the towers are approached, it was found necessary to break the grade with a step at each beam for about two-thirds of the distance, and the rise of these steps increases with the elevation. Spruce posts bolted to each beam support a $\frac{5}{8}$-in. galvanized wire rope handrail stretched tight. Frame towers at the middle of the land span carry sheaves for supporting a traveling rope at that point.

The main span, 1 600 ft. in length, is supported for a distance of 400 ft. out from each tower by means of 1-in. suspender rods; and the

PLATE V.
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CABLES OF SUSPENSION BRIDGE.

FIG. 1.—SADDLES FOR MAIN CABLE AND FOOTBRIDGE CABLES, ON TOWER.

FIG. 2.—PLACING SOCKET ON END OF ROPE.

remaining 800 ft., at the center of the span, rests directly on top of the cables, and is clamped to them by means of U-bolts passing between two 3 x 8-in. yellow pine floor beams bolted together. The suspender rods are hung to the clamps which bind the three ropes together. The lower ends of the suspenders are passed between the two floor beams at each point. The suspenders decrease in length as the distance from the tower increases, until they are discontinued, about 400 ft. out, where the beams rest directly on top of the cables, and are secured with U-bolts. Planking, 2 x 6-in., with 2-in. spaces between, laid across the floor beams, forms a footwalk. Cleats nailed crossways add to the security of the foothold.

The upper deck of the main span is supported on posts which rest directly upon the beams of the lower deck. These posts are capped with stringers which in turn carry beams to which is nailed the flooring similar to that on the lower deck. Handrails of ⅝-in. galvanized wire ropes are stretched on both sides of each walk.

The bridge is stiffened laterally by a cross-bracing of ⅝-in. rods.

Four 2¼-in. storm cables are suspended below the main span in the form of parabolas, the vertices of which are about 4 ft. below the vertices of the four footbridge cables. The storm cables cross each other between the center and the point where they are secured to the columns of the main tower about 50 ft. above high-water mark.

The land spans are held by guys attached to the steelwork of the truss forming the end spans of the main bridge.

Calculations Necessary for Design and Erection.

In considering the design of the footbridge, the following conditions and requirements were known:

(1) Length of center span, *i. e.*, horizontal distance between towers;

(2) Length of land spans, *i. e.*, horizontal distance between towers and anchorages;

(3) The exact position of the main cables when finished, and the exact position of each strand while being spun;

(4) The lower deck of the footbridge, when finished, must be about 4 ft. below the line of the main cable;

(5) The upper deck of the finished footbridge must be about 4 ft. below the line of a strand being spun;

(6) The floors of the two land spans when finished must be about 4 ft. below the line of the cable when in position;

(7) Each cable must be accessible from the footbridge at every point along its length.

The general design of the structure being decided upon, the preliminary step in the calculations was the location of the saddles on the towers and anchorages, in order to determine the exact span and deflection of the curves.

The cable passes over two saddles on each tower, placed 6 ft. 8 ins. on each side of the center of the tower.

Therefore: Length of main span = 1 600—2 × distance from center of tower to tangent of curve on saddle.

The position of the tangent point was found approximately by scaling the drawing of the saddle after the line of the cable was laid off on it approximately at the correct angle.

The span being thus determined, the versed sine of the curve had to be found. The elevation of the tangent point on the saddle was known. The elevation of the center of the main cable at the middle of the span was also known. The distance below the center of the main cable to the bottom of the footbridge cable = 4 ft. + the thickness of the flooring, floor beams and footbridge cable. The versed sine of the loaded footbridge cable was thus determined.

The curve of this cable, after the footbridge had been built upon it, would approximate a parabola, and it had been found by experience that the formula of the parabola might be used here with practically correct results.

The load upon the bridge, while not exactly uniform per horizontal foot, is nearly so. The load due to the weight of the cables themselves is greater near the towers where the slope is steeper, but this is partially balanced by the additional weight of the timber work at the center of the span due to the increased distance between the upper and lower decks at that point.

The total load on the main span is made up as follows:

Timber work	718 000	lbs.
Rods, bolts, clamps, etc	87 800	"
Cables	222 000	"
Storm-cables	55 500	"
Guys and storm-cable suspenders	19 500	"
Traveling rope	7 200	"
Total	1 110 000	"

PLATE VI.
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CABLES OF SUSPENSION BRIDGE.

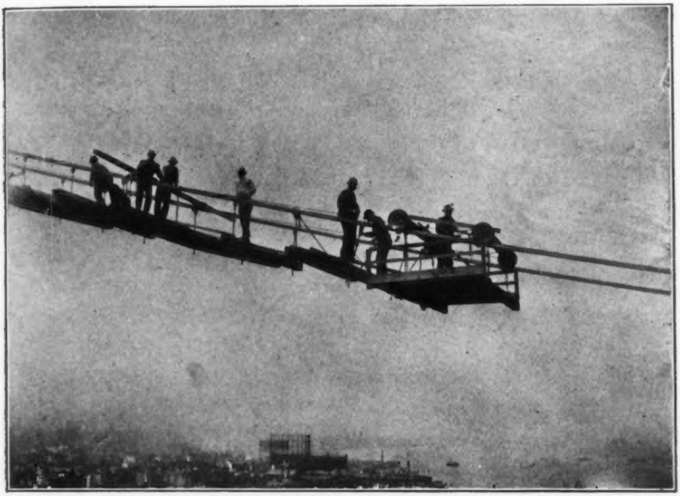


FIG. 1.—ERECTING MAIN SPAN OF FOOTBRIDGE FROM TRAVELER.

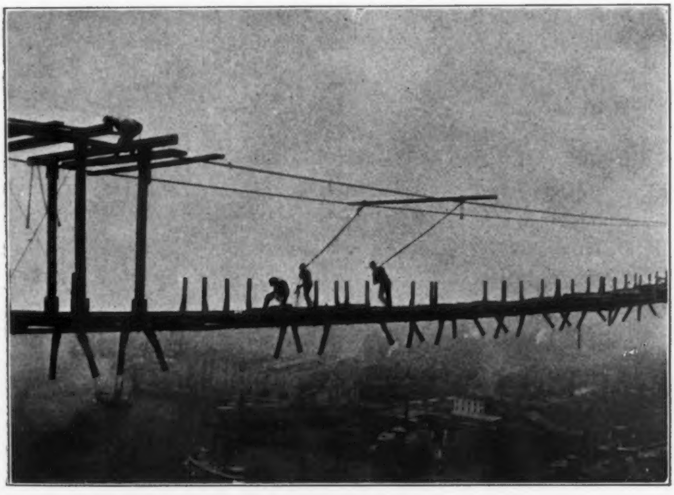


FIG. 2.—CONSTRUCTION OF UPPER DECK OF FOOTBRIDGE.

The load per foot $= \frac{\text{Total Load}}{\text{Span}}$; and the equation of the curve could thus be determined.

Next, it became necessary to find the equation of the curve of the main cable hanging unloaded. This cable, being of uniform section throughout its length, will hang in the curve of the catenary, the formula for which may be determined from the given position in which the cable is required to be suspended.

By computing an ordinate of the main cable and an ordinate of the footbridge cable at the same point, the position of the footbridge floor in reference to the footbridge cable was determined, and the operation was repeated at as many points as desirable. These ordinates were computed for every 50 ft., and the curve plotted on a scale of $\frac{1}{8}$ in. = 1 ft. After the two curves were laid out, the line of the floor of the lower deck was drawn, it being a constant vertical distance below the main cable.

After plotting these curves it was found that the line of the lower deck floor of the footbridge nearly coincided with the curve of the footbridge cable for a distance of 400 ft. on each side of the center of the span. The beams of the lower deck floor, therefore, were attached directly to the footbridge cables for that distance. The lengths of the suspenders for the remaining 400 ft. to each tower were computed from the known ordinates at every 50 ft., and the intermediate ones were found by interpolation.

The next step was to locate the position of the upper deck. The floor of the upper deck had to be a constant distance below the strand while it was being made, this position of the strand being due to its temporary support in a position somewhat higher than that which it will occupy ultimately. Supporting the strand on sheaves over the saddles and holding the end of it back of its final position in the anchorage causes this condition.

This temporary suspension of the strand will allow it to assume the form of the catenary, the formula for which can be found, and the curve plotted on the same sheet with the other curves. The position of the upper deck floor may then be plotted also. Comparing the ordinates of the curve of the upper deck floor with those of the lower deck floor, the lengths of the posts supporting the upper deck may be determined.

On the land spans the difference in elevation between the cable in

position and the strand being made was so small that a double deck was not necessary, as the strand in either position would be easily accessible from the same footwalk. The lengths of the suspenders in this span were readily computed by a comparison of the ordinates of the footbridge cable and the main cable in position.

Having settled upon the exact position and design of the finished footbridge, it became necessary to know in what position to suspend the unloaded cables, so that they might hang at the desired elevation after the weight of the structure had been placed upon them.

To arrive at this knowledge an inverse process of reasoning was necessary. It was assumed that the bridge was standing complete, with its cables in a known position and under a known tension. The length of the cable, under these known conditions, was computed from the equation of the parabola. Then it was assumed that all the load had been removed. The cables then would hang in the curve of the catenary. The average tension was then computed from the equation of that curve. The modulus of elasticity of the rope having been previously determined by tests to be about 17 000 000, the difference in length of the cable for this known difference in tension was then found. The new length being determined, and the curve known to be a catenary, its versed sine was readily computed. The elevation of the points of support being known, the elevation of the vertex of the curve was found.

To suspend the cable at the correct elevation a leveling instrument was set up in one of the towers at exactly the elevation at which the lowest point of the cable should hang.

For the adjustment of the cables in the two land spans a similar process of reasoning was used to find the curve in which the unloaded cables would hang; but, as the vertex of the curve lies outside of the points of support of the catenary, the method of adjusting the cable by means of the level could not be used. In this case the transit had to be used. It was set up at a known point between the two supports and in such a position that a line of sight could be taken tangent to the curve of the cable.

The equation of the tangent which passes through the point at which the instrument was to be stationed was found by previous calculation. The inclination of this line being known, the transit was set up and the telescope inclined at the correct angle of the tangent. In

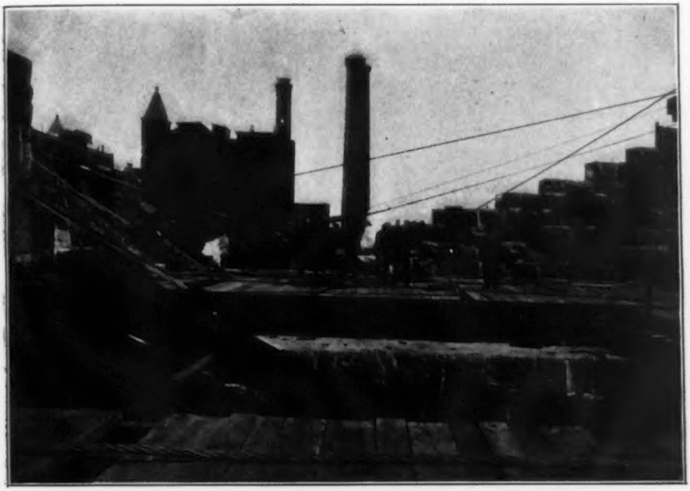


FIG. 1.—TAKING TENSION ON TACKLE FOR FINAL PULL ON FOOTBRIDGE CABLE.

FIG. 2.—LAND SPAN OF FOOTBRIDGE AS SEEN FROM BROOKLYN ANCHORAGE.

adjusting the cable it was only necessary to hang it so that its underside was tangent to the line of sight of the transit.

These calculations were all based upon a mean temperature. However, as the temperature plays an important part in the position of a cable of such length, it became necessary to make due allowance for such changes. The specified position of the main cable unloaded formed the basis for all calculations as to the position of the footbridge. This position is given for a mean temperature of 60° Fahr.

The first figures obtained for adjusting the cables of the footbridge, therefore, had to be tabulated for a temperature of 60° Fahr.

The elevation for which to adjust the cable in the main span for this temperature had already been determined. If it is assumed that the temperature on the day the cable was to be adjusted would be 1° higher, the increase in temperature would cause an increase in length, but the increase in length would also cause a decrease in tension. This decrease in tension had to be taken into account in determining the amount of stretch due to tension. The true length was thus found for the new temperature, and the deflection computed and tabulated. This was done for all temperatures which were likely to occur during the time of making the adjustment. In the case of the land spans a different tangent was found for the various temperatures, and its angle tabulated, to be used as required.

Erection of the Cables.

The footbridge cable ropes were shipped by rail from Trenton, N. J., where they were made in the shops of John A. Roebling's Sons Company. They were transferred to the foot of the Manhattan tower on the deck of the largest tug of the Merritt, Chapman Wrecking Company. Each rope was $2\frac{1}{4}$ ins. in diameter, and about 3 020 ft. long. It came on a reel 7 ft. in diameter and 4 ft. wide, the whole weighing about 14 tons. A float, 180 ft. long and 25 ft. wide, previously used for the transfer of freight cars, was moored at the foot of the Manhattan tower. Three reels at a time were placed upon this float, mounted on stands bolted to the deck and placed so that the ropes could be reeled off over the side. Each reel was provided with a cast-iron brake rim over one flange of the drum, around which was passed two turns of $1\frac{1}{2}$-in. manila line, one end being secured to the deck and the other held by hand.

The end of each rope was unfastened from the reel, raised over the tower, drawn back to the anchorage and secured there. This was accomplished in the following manner. The fall line from a derrick on top of the tower, operated by a hoisting engine on the float, was attached to the rope by means of a clamp about 60 ft. from its end. The end of the rope was then hoisted to the top of the tower. As the rope unwound from the reel, the loose part passed over the saddles and was placed in temporary rollers mounted on them. Plate V, Fig. 1, shows the saddles for the main cable and for the footbridge cables, one footbridge rope being in position and the other in the rollers ready to be pulled over. Another clamp was now attached about 10 ft. from the end of the rope, to which was connected a long line running back to the anchorage and operated by means of another hoisting engine located on the end-span truss about half way between the anchorage and the tower. After the tension had been taken on this "runner" the fall line on the derrick was relieved of its load, and the clamp loosened on the rope so that it would slide down easily to the bottom of the tower. There it was once more clamped to the rope, and both engines were operated, one lifting it and the other pulling the end back to the anchorage. It was necessary to readjust the lifting clamp each time it reached the top of the tower, until the end of the rope had arrived at the anchorage.

The socket was next put on and the connection made to the short piece of rope previously secured to the anchorage girder. This operation was repeated until all three ropes forming a cable were similarly placed Each in turn was removed from the rollers on top of the saddles and lowered into place in the saddle.

The next operation was to get these ropes across the river. This was accomplished with the assistance of three powerful tugboats, which towed the float across the river at slack tide. This time was selected because the course could be more easily controlled and because at that time vessels could be more easily handled and kept out of the way. The float was towed sideways, with a tug at each end and one ahead to pull. The operation of towing the float across the river is shown in Plate IV, Fig. 2. The course was kept as nearly as possible in a straight line between the two towers, and, as it was traversed, the reels were allowed to pay off the rope, which sank and lay upon the bed of the river, the ropes having been previously lashed to the bottom of the Manhattan tower.

Fig. 1.—Land Span of Footbridge during Construction.

Fig. 2.—View of Main Span of Footbridge, from Traveler.

The trip across the river occupied about 14 minutes, during which time great care had to be exercised that the reels did not pay off too rapidly. When the Brooklyn side was reached, the float was moored to the tower. It was then found that about 400 ft. of rope remained on each reel. This was unreeled, one reel at a time, and laid along the full length of the deck of the float. About 60 ft. from the end of the rope a clamp was secured, to which the fall line from a derrick on the tower was attached. The engine on the float operated the line and hoisted the end of the rope to the top of the tower. The loose end was next passed over the tower and placed on the rollers on top of the footbridge saddles. Another clamp was placed near the end of the rope, to which was attached one end of a three-part $\frac{5}{8}$-in., wire-rope tackle, which reached to the Brooklyn anchorage, and was operated by a 60-H.-P. hoisting engine. The clamp was removed from the river side of the rope as it lay over the saddles. The lashings which held it near to the foot of the Manhattan tower were cut, and everything was then in readiness to raise the rope from the bed of the river, and stretch it from tower to tower. Two government patrol boats guarded the river to prevent passing craft from interfering with the operation, which was performed at slack tide.

At a signal from a steam whistle on the float (the signal also serving as a warning to passing boats), the engine on the Brooklyn anchorage began to pull the rope over the tower, and gradually drew it from the river bed until it had been raised nearly to its correct position. This operation consumed from 4 to 6 minutes. By this time the upper block of the tackle had traveled to the Brooklyn anchorage. The main span was now so nearly balanced with the Manhattan land span that workmen on top of the Manhattan tower, were able, by means of a set of hand tackle, to pull the rope over the tower until it was correctly adjusted.

A man with a transit stationed at the correct point below the curve fixed the position of the rope according to the temperature. Once adjusted, the men on the tower marked the rope in its correct position, and secured it against future movement.

At the same time an additional clamp was being placed on the end of the rope which had been pulled down to the Brooklyn anchorage. This was placed a little above the other clamp, and, attached to the former, was a nine-part manila tackle with its other end secured to

the short rope fastened to the anchorage girder. A tension was now taken on the manila line by the engine, and the tension on the wire-rope tackle relaxed and the clamp removed. The operation of taking the tension on the nine-part tackle for the final pull on the footbridge cable rope is shown in Plate VII, Fig. 1. In the meantime the rope was lowered into the saddle on the Brooklyn tower, and everything was made ready for the final pull.

A man with a level stationed in the Brooklyn tower at the correct elevation according to the temperature, gave the signals to raise or lower the rope until it hung at the correct height over the middle of the river. When this was adjusted, the position of the rope was marked on the Brooklyn tower, and secured against future disturbance.

The next step was to adjust the Brooklyn land span. This was done, as on the Manhattan side, by means of a transit set at the correct angle. The rope was lowered or raised to the correct position, which was marked on the saddle at the face of the anchorage as soon as adjusted. The rope was then measured and cut off, the socket put on, and connected to the short rope fastened to the anchorage girder. See Fig. 2, Plate V.

This completed the stretching of one of the footbridge cable ropes. The operation was repeated until twelve ropes, assembled into four cables, were similarly suspended. The adjustment of the subsequent ropes was made by comparison with those previously adjusted, but all were finally checked independently.

Erection of the Superstructure.

The placing of the timber work was begun at each tower, and worked downward toward the center on the main span, and toward the anchorages on the land spans. Each piece was framed and marked as to its location in the structure before being hoisted from the ground. Two travelers for the main span were first erected at each tower, and two for each land span. In each case a traveler ran on two cables by means of grooved wheels connected by an axle to which was suspended a platform large enough to accommodate four or five men. The cable bands, suspender rods and floor beams were attached to the cable by the men on the traveler. The traveler was allowed to run down the cable in advance of the work. The

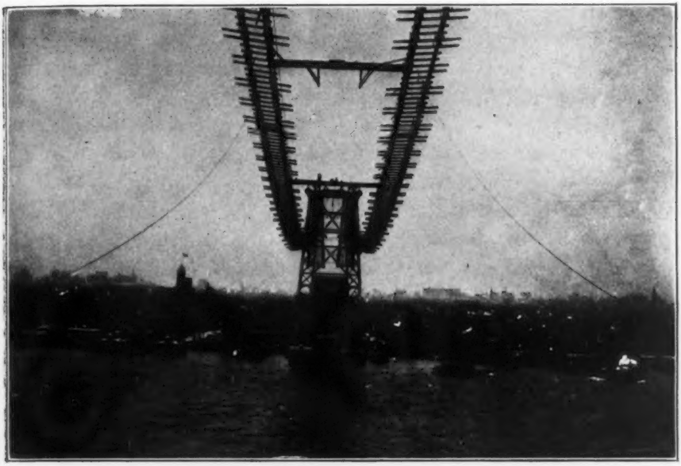


FIG. 1.—LOWER DECK OF MAIN SPAN OF FOOTBRIDGE.

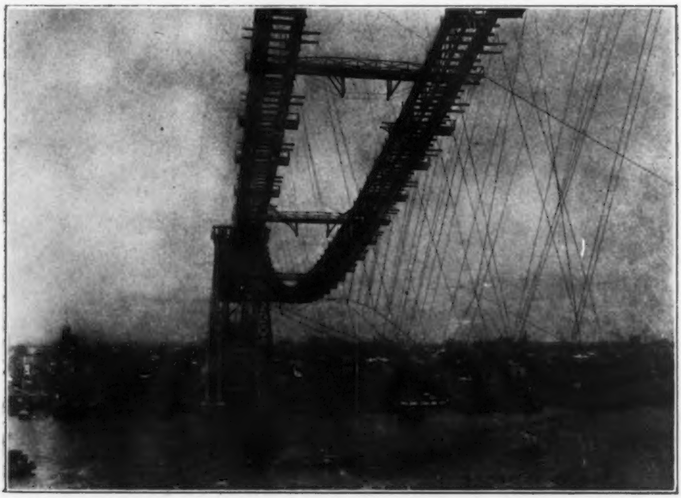


FIG. 2.—MAIN SPAN OF FOOTBRIDGE WITH STORM CABLES AND GUYS IN PLACE.

traveler is shown in Plate VI, Fig. 1. Another gang followed up the traveler and laid planks across the floor beams as fast as they were swung into place. Plate VIII, Fig. 2, is a view from the traveler on the main span, looking back at the work just placed. In the main span the material was carried down from the towers by hand, but on the land span each bent was framed complete on the truss below and hoisted, and the men on the traveler connected it to the cables. Plate VIII, Fig. 1, is a view of one of the land spans during construction, in which the travelers have advanced about half way to the anchorage. It was necessary to keep the work on the two sides of the tower well balanced, so as not to cause the cables to slip over the saddles from undue loading in one span.

The travelers advanced toward the center of the main span simultaneously from each tower, so that the meeting point was at the center. When this point was reached the travelers were taken apart and carried back over the footwalk to the towers.

A view of the main span from below, at the level of the roadway in the tower, is shown in Plate IX, Fig. 1.

The land-span travelers were run down to the anchorages, and taken apart there. Handrail posts were next bolted to the floor beams, and a ⅝-in. galvanized wire rope for a handrail was stretched along the full length of each footwalk.

Before commencing to erect the upper deck of the main span four additional 2¼-in. ropes were suspended. These ropes were drawn across the bridge from the Manhattan tower, the reel being placed at the foot of the tower. This pulling was done by an engine on the Brooklyn anchorage with a ⅝-in. plough-steel line, passing over the Brooklyn tower and running on the sheaves on top of the footbridge saddles. Each of the four ropes was hung just above one of the footbridge cables, and rested in the same saddle at the top of the tower. These ropes were suspended at a certain elevation, so as to be just below the floor beams of the upper deck, and carry their proportion of the load of the bridge. In each land span they were drawn down and clamped to the other three ropes forming a cable. Plate VII, Fig. 2, is a view of the land span of the footbridge, as seen from the Brooklyn anchorage, as it passes through the steelwork of the end span.

The erection of the framework for the upper deck was begun at the center of the main span and carried toward the towers. Plate X, Fig. 1, is a view of the main span, showing the erection of the upper deck. The material was run down on top of the cable by bridgemen who held each end with a line and ran along the footwalk with each piece, as shown in Plate VI, Fig. 2. When the posts, stringers and floor beams were in place, the 2 x 6-in. flooring was laid over the beams, beginning at the towers and working toward the center. The flooring on the land spans was similarly laid. Handrails of $\frac{5}{8}$-in. galvanized wire rope were stretched on each side of every footwalk, and secured to the handrail posts with staples.

Half way between each anchorage and tower and at three points along the main span were erected frame towers about 12 ft. high. On these were supporting sheaves over which ran a traveling rope.

Storm Cables.

The four $2\frac{1}{4}$-in. storm cables previously described, each about 1 700 ft. long, were delivered on reels at the foot of the Manhattan tower. The free end was taken to the top of the tower, passing up the land side and crossing over the top toward the river on the sheaves on top of the footbridge saddles. The rope was now drawn across the footbridge in the same way as were the cables for the upper deck. They were laid across the lower deck beams and in such a manner that they could be easily pushed overboard. While in this position a suspender of $\frac{1}{2}$-in. galvanized wire rope was fastened every 40 ft. The other end of the suspender was attached to the foot-bridge cables. The length of each had been regulated by previous calculations, so that when the storm cable became suspended by them it would hang in the form of a parabola.

When all the suspenders were properly connected the cable was lowered overboard by means of blocks and falls at numerous points until it was hung on the suspenders. The two ends were then pulled in to each tower, and sockets placed upon them. Each was secured to one of the main posts of the tower and screwed up to the proper tension by means of long U-bolts. A view of the main span, with the storm cables and guys in place, is shown in Plate IX, Fig. 2. The footbridge was secured additionally by means of long guy ropes which ran from the main span down to the towers.

FIG. 1.—MAIN SPAN OF FOOTBRIDGE, SHOWING ERECTION OF UPPER DECK.

FIG. 2.—COMPLETED FOOTBRIDGE.

Plate X, Fig. 2, is a view of the completed footbridge as seen from Brooklyn, just back of the anchorage. When the structure was completed, observations were made to determine how near the bridge came to occupying the position for which it was finally intended. The result of the observations showed it to be within a very few inches of the calculated position, which was a great source of gratification to the engineers in charge, as it proved the correctness of the assumptions, the exactness of the intricate calculations involved, and the accuracy of the instrument work.

When the last wire in each main cable had been laid and the last strand adjusted, the usefulness of the upper deck of the footbridge was at an end. The whole of the upper deck work was then removed, the flooring being transferred to the lower deck and the other timber work being kept on the span, so as not to change the loading and consequently the position of the bridge. The upper cables were tied to the lower ones by long U-bolts, so as to make each carry a proportion of the load.

The work of putting on the cable bands and cable covering, and hanging the suspenders, can now be completed from the lower deck of the bridge.

The expectations of what the footbridge would accomplish have not been over-estimated, for it has permitted a saving of much time, made the work safer, and afforded a means of building the cables as nearly perfect as it is within human possibility to do. The erection of such a bridge may be usually considered a very hazardous occupation, but in this case not a single accident occurred.

Reference to cable making, except in explanation of the functions of the footbridge, has been purposely omitted in this paper, as that is a subject in itself.

DISCUSSION.

Mr. Hildenbrand.

W. Hildenbrand, M. Am. Soc. C. E.—Mr. Harby has given a very clear and interesting description of the building of that airy structure known as the temporary footbridge over the East River, which has been in position for almost two years, and during this time, in spite of its apparent unsafety, has braved many storms. It has now nearly fulfilled its purposes, and soon will be an object of the past; in fact, the work of dismantling it commenced several weeks ago.

This footbridge was a structure of considerable magnitude, presenting some novel and unique features, and it is hoped that the presentation of this paper may have contributed at least a mite to the knowledge of all members of this Society.

It is presumed that many will feel partially disappointed at a certain seeming incompleteness of Mr. Harby's paper, and may ask why the means to accomplish a certain object has been presented and not the object itself. If it were necessary to erect such a large and expensive structure merely to be used as a convenience for doing some other engineering work, it would be natural to suppose that the latter must be of great magnitude or of more importance; and if it were worth while to describe the scaffold, so to speak, why not the work to be created with the help of this scaffold.

In other words, it may be asked why this paper is not extended so as to go on with the description of the big cables, and of the method of making them.

In answer to these questions, the speaker wishes to say that Mr. Harby's paper is the introduction to another paper: "On the Making of the Cables of the New East River Bridge," which is now in course of preparation by the speaker, and which he will have the honor to submit as soon as it can be finished.

The footbridge is really a part of the cable making, but in this particular case it was of such prominence that it was arranged between Mr. Harby and the speaker to divide the labors, and that the latter's paper should be an immediate sequel to the present discourse; but, unfortunately, the preparation of this paper has been delayed, and it will be several weeks before it can be presented.

The speaker thinks that the paper on the footbridge is appreciated, and feels personally indebted to the author, because his illustrations have lightened considerably the labors still before the speaker.

On page 171, Mr. Harby says:

"After plotting these curves it was found that the line of the lower deck floor of the footbridge nearly coincided with the curve of the footbridge cable for a distance of 400 ft. on each side of the center of the span."

This sentence gives the impression that the calculations had been made and plotted, and, in consequence of having found a certain result, the conclusion had been reached that the flooring should be put on top of the cable. The fact is that the bridge was designed first and then the calculations were made for finding a cable curve, which, for 200 ft. on each side of the center of the span, would touch the underside of the floor platform, exactly as it had been designed. Mr. Hildenbrand.

On page 172, in giving a description of the different calculations, Mr. Harby says several times that the ordinates and the length of the suspenders, etc., were "readily" computed. The speaker likes Mr. Harby's optimistic way of expressing himself. The fact is, it took a good many weeks and months of diligent labor for the preparation of long and intricate formulas; and when it came to the actual numerical calculations, Mr. Harby and the speaker were obliged to spend many days, working until after midnight, to come to the results ; so that if he calls this "readily computed," the speaker would like to see those computations that are not "readily computed."

Later on, Mr. Harby says: "A man with a transit stationed at the correct point below the curve fixed the position of the rope according to the temperature."

The speaker desires to state that Mr. Harby, himself, was that man, and it was not a very enviable position. He had to stand on an improvised platform erected on the beams of the side spans, with no floor under it, and he had just enough room to take half a step back and look through the telescope, otherwise he looked down 100 ft. on either side of him; and this he had to do in a temperature of 18 or 20°, sometimes in a cold drenching rain. His work was accurate. After the footbridge was completed it occupied precisely the position which was fixed by calculation.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 940.

THE MAINTENANCE OF ASPHALT STREETS.*

By James N. Hazlehurst, M. Am. Soc. C. E.

With Discussion by Messrs. S. Whinery, Nelson P. Lewis, S. C. Thompson, J. M. Evans, W. Boardman Reed and James N. Hazlehurst.

Since the impetus given to improved road making, through the genius of Macadam and Telford, nothing has quickened the interest or added more to satisfactory results in highway construction than the introduction of natural bituminous concrete or asphalt as a surfacing material. Appearing as a practical utility simultaneously in Berlin and in Washington, about 1876, its remarkable popularity is attested by a bulletin issued by the United States Department of Labor in 1900, reporting 30 203 946 sq. yds. of asphalt laid in 129 cities in this country having populations of 30 000 inhabitants. When it is recollected that this represents an outlay of approximately $75 000 000, in initial cost and expended during the last 25 years, the enormous proportions of this industry are apparent.

Notwithstanding the great popularity of this class of pavement, both in this country and abroad, its introduction has been attended with many expensive and disastrous failures, as a reason for which, E. Kuichling, M. Am. Soc. C. E., in a recent article, cites the following:

"It may also be mentioned that rock asphalt for street paving purposes was first introduced in Paris in 1854, and that its advantages

* Presented at the meeting of September 17th, 1902.

were promptly recognized by Baron Hausmann, Napoleon III's famous Minister of Public Works. Not only did it afford the means of securing a handsome, sanitary and comparatively noiseless pavement, but its continuity and difficulty of removal made it of little use for the formation of barricades by the turbulent element of the population, and thus contributed greatly to the preservation of law and order by the municipal authorities. Its use in most of the principal streets and avenues soon followed, but with the increased demand came competition and numerous attempts to reduce the cost by the substitution of inferior materials and methods, until finally the quality became so poor as to lead to early disintegration. This result became generally noticeable in Paris about 1880, and in consequence of the inability of the contractors to make proper repairs, asphalt pavements fell into serious disrepute.

"In relation thereto, M. Leon Malo, the well-known French paving expert, and director of the rock asphalt mines at Seyssel, which are controlled by the General Asphalt Company of France, states that up to 1876, all the asphalt pavements of Paris were built and maintained by said company under contract, and gave entire satisfaction. In that year, however, the contracts expired, and the municipal authorities yielded to the pressure brought by rival paving corporations to open the work of construction and maintenance to general competition, and award the contracts to the lowest bidder. The consequence was that the work was taken at low prices by inexperienced firms, and performed improperly, as mentioned above. In the course of five years over 240 000 sq. yds. of the inferior asphalt pavement crumbled, and the contractors went into bankruptcy, leaving repairs of enormous extent undone and creating widespread prejudice against this kind of pavement. The authorities then realized that such work could not be done in a haphazard manner, and thereafter gave contracts only to the most experienced and responsible firms, under strict inspection, and limiting the source of the bituminous rock to a few quarries of established reputation."

The experience of the French capital is not an isolated case, many cities on both sides of the Atlantic having successively faced and decided, each for itself, the advisability of restricted competition or untried materials.

In engineering works, "what is wanted" is generally so clearly understood that there is no difficulty either in readily specifying or exactly following the requirements. The strength, durability and composition of the great mass of materials entering into constructive engineering works have, for the most part, been long and well understood, and their exact values determined experimentally and prac-

tically under a wide range of conditions and tests, but to this rule there are certain notable exceptions, where materials are used singly or in combination with others for certain important work, but where the individual character and resulting combination is made without sufficient scientific knowledge to make certain prediction of successful use.

Such a material is bitumen, the essential base of all asphalt pavements. Bitumen, in mineralogy, is defined as "a hydrocarbon mixture, of mineral occurrence, whether solid, liquid or gaseous," and, in line with this, the Supreme Court of the United States has ruled that natural gas is a true bitumen; other chemists and mineralogists are inclined to include "any and all hydrocarbons, whether natural or artificial, provided they be soluble in carbon bisulphide; hence, in specifications where the percentage of bitumen is a ruling consideration, either natural gas or coal-tar would be equally suitable for the production of an asphalt pavement.

Asphalt is a hard, natural bitumen, and is found in Nature in great deposits, such as the pitch lake at Trinidad, or impregnating both lime and sand rock. Not only does uncertainty exist in the classification of bitumens, but, in analytical chemistry and physical test, the differentiation of asphalt and coal-tar is most uncertain. To the chemist, the diamond and the lump of coal are both carbon, while coal-tar and natural asphalt are bitumens, almost impossible to determine, the one from the other, except by the smell.

The wearing surface of the asphalt street should be a true concrete, in which sand is the aggregate, while carbonate of lime dust and the asphalt form the matrix. As with hydraulic concrete, it is important that all voids in the mass be filled, and that asphalt, the cementing ingredient, shall be of proper composition and quality, to the end that the concrete thus formed is durable, tenacious and elastic. While all asphalt is bitumen, all bitumen is not suitable for an asphalt pavement; it may be too brittle or may lack the requisite cementing properties, in which case the pavement necessarily disintegrates and fails. Again, even with the best of materials, the composition may be spoiled in the making, too much of the limestone dust making the pavement hard and brittle, while too great a quantity of asphalt cement will make a soft and yielding surface in summer, sometimes being so soft as to mire teams, in either case making a renewal of the wearing sur-

face of the pavement a necessity; so that it would seem that the surest way of securing satisfactory results and determining the merits of an asphalt pavement is to give it a trial, requiring considerable time, and to award the execution of the work to experienced asphalt contractors and road builders.

In view of the negative value of tests, except that of long experience, the only specifications likely to produce reliable and exact results, are those which incorporate in their requirements formulas and methods adapted to materials which have previously been used successfully, as made evident by from fifteen to twenty years of steady use under ordinary street traffic; but even the adoption of such well-tried and successful materials and methods is not a positive guaranty of the merit of the asphalt proposed to be laid, for careless manipulation and unaccustomed climatic conditions may make an inferior pavement of materials which have been used successfully elsewhere, while restricting competition to those materials the fitness of which has been established by long use necessarily reduces competition, offering inducement to collusion, combination and fraud; besides, there is lost the possibility of introducing through experiment a material superior to any so far presented for service.

Such considerations suggest the necessity for the broadest competition, and the free exercise of this principle by municipalities is most commendable and, in general, is certainly the true public policy. However, it is equally true that the taxpayer should be at all times protected against the possibility of expensive failure, where inferior or untried materials and methods are permitted to compete upon the same basis with those which from long experience have fully demonstrated their fitness for particular use; and it is obvious that the safe course, under such conditions, is to insist that the burden of responsibility and the cost of full and free experiment be placed where it properly belongs, upon the promoter of the new material.

Were it not for a legal obstacle, to be hereinafter discussed, it would be a simple solution of the difficulty to require a solvent bond for performance and a long period of maintenance.

Unfortunately, any long-term guaranty is likely to be subject to sharp and repeated attacks before legal tribunals, with every chance of having assessments defeated or declared invalid because of the incorporation, in paving contracts, of a clause requiring the con-

tractor to guarantee and keep in repair, for a long-term period, the work executed by him.

The contention has frequently been sustained by the highest courts of the land that such a requirement, unless clearly set forth and intended by the act providing for paving assessments, works an unexpected hardship upon the abutting property owner, who is not only compelled to pay for all or a proportion of the cost of paving in front of his property, but is charged besides with the maintenance of the street in good order for a definite period of years under the so-called "guaranty" clause of the contract, which is, in fact, only a maintenance charge in disguise; an expense which should clearly be assessed against the taxpayer at large, responsible for the proper care and repair of all streets and public places, there being no sanction in the statute or act for putting this expense upon the property owner.

The Supreme Court of the State of Louisiana, on May 29th, 1899, handed down a far-reaching decision, which, in part, held that:

"The maintenance clause in the contract, by which plaintiff company binds itself to keep the street in good order and condition for a term of five years, must be construed with reference to the specifications for the work, and the bid of the plaintiff thereon."

The latter contained the guaranty:

"That the work would be constructed in such manner that the same would endure without requiring repairs for five years, but that, if repairs should become necessary, plaintiff company would make same at its expense. It thus appears that plaintiff's undertaking was to lay paving consisting of such materials, and put down in such manner, as to endure for five years without repair, and it guaranteed its work to be of such character. If not of such character, the loss would fall on the company, at whose expense the repairing needed within five years would be done. This clause is not legally objectionable. It is regarded as simply a guaranty of the quality of the work contracted to be done, and does not render the contract void, as increasing the burden of abutting property owners by requiring them to pay for keeping the pavement in repair for the period of five years after its completion. It is an incident of the contract, not an independent undertaking."

There must be some point, however, where such an obligation for maintenance does become an "independent undertaking," and that point is conceived to be that period of time within which the well-laid asphalt pavement should continue without requiring repairs incident to the wear of ordinary traffic.

A consideration of the department reports of some of the larger cities shows that in the great majority of cases the maintenance of asphalt pavements is provided for by the contractor under the "guaranty" clause for a period of about five years, hence the impossibility of obtaining from the records an approximation of the cost of providing for necessary repairs during such time, but, from reliable authority, it seems certain that the contractor never anticipates the necessity of repair of asphalt pavements, except for accidental cause, during a five-year term; hence that period might properly represent that specific time when maintenance becomes more than "an incident" of the contract cost of paving.

Referring to the experience of Paris, it would appear that about the same period was required before inferior pavements disintegrated to a point necessitating removal; hence it is questionable whether that period of time taken to represent a reasonable guaranty is sufficient to test fully the life expectancy of the asphalt pavement. If this contention is accepted as true, the wisdom of the German municipalities is unquestioned. Their contract requirements for asphalt pavements being that the new pavement shall be kept in perfect repair for a period of nineteen years, beginning on April 1st of the year following the completion of the work. During the first four years the contractor receives no compensation whatever for necessary repairs, but for the remaining term of fifteen years he is paid at the rate of 10 cents per square yard for the entire area under contract.

While the practice in the United States is to omit altogether any provision for long-term guaranties by the original contractor for asphalt pavements, in a few instances, subsequent and supplemental contracts for such repairs have been entered into by municipalities, and include periods ranging from five to ten years in addition to the time of the original guaranty.

The City of Omaha, Neb., has made such a contract, for ten years, at the rate of 8 cents per square yard per annum; Denver pays 10 cents per square yard for a like arrangement, while Cincinnati, subdividing the term into two periods of five years, has contracted for the repair of its asphalt pavements at 7½ cents for the first five years and 14 cents per square yard for the last five years, in all, fifteen years of guaranty or maintenance.

Under the European system, providing for the maintenance of

asphalt streets at a fixed sum per square yard per year, such bonus, payable annually, depends only upon the extent of the area under contract and not upon its legitimate and natural repairs, the effect being that the contractor receives a sum for repairs whether he earns it or not, the sum paid representing the premium upon an insurance policy rather than an expenditure for a real necessity.

The practice of a few American cities, in providing for the maintenance of streets by subsequent additional and supplemental contracts, not necessarily with the original contractor or for identical material, might be so amended as to provide that the first contractor should continue to keep his work in repair, with materials previously used, the payment for which being based upon original prices, and due only where there is a real demand for repairs and a consequent outlay.

From the department reports of Washington and Buffalo, the cost of repairs and maintenance was guaranteed by the contractors for five years. The subsequent expense to each city is averaged as follows:

WASHINGTON.

First period of five years....	0	cents per square yard per year.
Second " "	2.9	" " "
Third " "	7.3	" " "

BUFFALO.

First period of five years....	0	cents per square yard per year.
Second " " "	6	" " " "
Third " " "	4.8	" " " "

Accepting the figures showing the cost of maintaining pavements in Washington as a basis, and rounding them out to 3 cents and 8 cents, respectively, and accepting the logical consequence of the preceding argument, it would seem perfectly possible and proper to draft specifications so as to permit open and broad competition, under a sufficient and proper guaranty, and in such manner as to comply with the rulings of the courts in the matter of maintenance, by stipulating that the contractor should lay the pavement at a price to be agreed upon and to be assessed against the responsible parties under the act providing for pavement assessments, but requiring the contractor to provide a solvent bond, not only for performance, but

for the maintenance of the work in good condition for a term of years, and for compensation which, from the record of other cities, might seem proper and reasonable. The cost of maintenance, thus determined and agreed upon in advance, should be paid annually, only when earned, to the contractor, out of the general city funds, for all legitimate repairs, and at the price previously agreed upon, but any excess of cost, greater than that which the record of other cities has shown to be a proper and just amount of repair work for well-laid asphalt pavement, should be assessed against, and paid for, by the original contractor.

For the reason that neither the chemist, mineralogist nor engineer can specify with certainty the character, amount or composition of an asphalt pavement, and as results and not precise methods are sought, it would seem reasonable to relieve the contractor of specific limitations, while requiring that the entire responsibility for the success or failure of the pavement be borne by him, through stipulation with security that the particular pavement shall last as long and require as few repairs as other well-laid asphalt pavements; and that the reasonable cost of legitimate and necessary repairs should be paid annually, after five years, to the contractor, by the city at large; while any sum expended in excess for necessary repairs should be a charge against the contractor.

In line with the preceding argument, the writer drafted for the Department of Public Works, of Mobile, Ala., a set of specifications from which the following is taken:

Specifications for Asphalt Pavements.

"*Wearing Surface.*—Upon a concrete foundation, or 'binder course,' previously prepared, there shall be laid a wearing surface of asphaltic concrete, composed of natural bitumen, silica and carbonate of lime, of such proportions and composition, and mixed according to such formula, as may be recommended by the contractor, who shall furnish, with his proposal, the chemical analysis of the asphalt to be used on the work; also, a statement of the ingredients and proportions thereof, and the method of mixing and laying. This material shall be spread and rolled to a finished depth or thickness of not less than 1½ ins. where a binder course is used, and not less than 2 ins. where such course is omitted, and in such manner and by such means as the contractor may deem expedient or likely to produce the best results.

"Concrete made of asphaltic cement and clean gravel may be used to form the binder course; its thickness shall be considered as part of the depth specified for cement concrete, and will be paid for as such.

"For a period of fifteen years, immediately following the acceptance of the work contemplated under these specifications, the cost of maintaining the pavement in good condition shall be guaranteed by the contractor, not to exceed the following rate for each square yard of pavement for the following periods:

"For the first five years, cost per yard for each year...... 0 cents.

"For the second five years, cost per yard for each year... 3 "

"For the third five years, cost per yard for each year.... 8 "

"The maintenance of the pavement in good condition during the guaranty period contemplates, and is intended to provide only for, repairs which may be necessary by reason of defects in the wearing surface of the pavement, made apparent by conditions of weather and traffic. Noticeable irregularities of the wearing surface; cracks exceeding ¼ in., and apparent disintegration, all extending over more than 1 sq. ft. of surface, shall be considered cause and necessity for repair, which shall be made and estimated as follows:

"*Guaranteed Annual Cost of Maintenance of Pavements.*—After a pavement has been laid and accepted, and the cost of maintenance guaranteed, at any time during the term of such guaranty, when, in the opinion of the Board of Public Works, and in accordance with the foregoing requirements as to maintenance, the necessity exists for repairing any portion of the pavement, upon notice from the Board, and within fifteen days thereafter, the contractor shall take up, relay or repair such portion of the pavement as may have been designated, and shall repair or relay the same with materials and according to methods prescribed for the original work, and to the satisfaction and acceptance of the Board of Public Works.

"For the purpose of estimating the value of such repair work, in addition to the contract price per yard as established by the original bid, there shall be allowed an extra amount for such repairs, as follows:

For each separate amount, as ordered, less than 20 yds., 20 per cent.

For each separate amount, as ordered, more than 20 and less than 50 yds., 15 per cent.

For each separate amount, as ordered, in excess of 50 yds., 10 per cent.

"Should the cost of repairs, as estimated above, exceed in any one year the sum for which the contractor has guaranteed that such pavement could be maintained, any additional work, ordered by the Board of Public Works, shall be done by the contractor, under the terms of the specifications, and without cost to the city.

"If, in the opinion of the Board of Public Works, there should arise the necessity for removing, repairing and replacing any section of the pavement for the laying of water, gas or other mains, or for the repair of the same, or for any purpose whatsoever, upon the written order of the President of the Board of Public Works, the pavement shall be opened, replaced or repaired by the contractor, in the manner and under the terms provided for repair work, but the cost of such work shall not be charged to the contractor, or considered as a part of his guaranty."

Under the provisions of the foregoing specifications and the penalty expressed in a bond in the sum of $15 000, contractors for asphalt paving have recently undertaken some 17 000 sq. yds. in the City of Mobile, Ala., for the following prices:

A.—Construction of 6-in. concrete foundations, including grading..............	$0.65	per square yard.
B.—Paving with asphalt..................	1.15	" "

DISCUSSION.

Mr. Whinery. S. WHINERY, M. Am. Soc. C. E.—The question of the wisdom of requiring a long-time guaranty on public work, and particularly on street pavements, has not received the attention its importance warrants. Whether such guaranties, on the whole, are beneficial to the interests of a city, or otherwise, certainly admits of arguments on both sides. The arguments on the negative side, apparently, have not been worked out carefully or pursued to ultimate consequences or conclusions.

The speaker does not include in this discussion those short-period guaranties to the effect that any defective materials or workmanship appearing within a period of six months or one year shall be made good by the contractor, nor to those which require that machinery or plant shall accomplish certain stipulated results. There can be no question about the propriety and the advantage of such guaranties. Nor is it intended to discuss that class of guaranties which require that work constructed with comparatively unknown materials or performed by unusual methods shall fulfill the requirements stipulated. Without attempting at this time to enter upon a full discussion of the wisdom of requiring long-period guaranties upon street pavements, reference may be made to some of the more important arguments against the practice.

The general theory upon which such guaranties are based is that the responsibility for the good and sufficient quality of the materials used, and of the workmanship and skill, is thrown upon the contractor under conditions which he cannot afford to disregard. In short, the object is to shift responsibility for the character of the work from the city authorities to the contractor. No one can object to this as a principle. The city is undoubtedly justified in taking any reasonable measures that will compel the contractor to comply fairly and fully with the requirements of his contract. The only question, therefore, that may be raised, is, whether, on the whole, such guaranties, in their practical workings, result, or can be made to result, to the advantage of the city. This embraces the question whether such guaranties, as they are usually framed, can be legally enforced, under the conditions that usually prevail in street-paving contracts.

In the first place, assuming that such guaranties are entirely legal, their value must depend upon the ability of the city to enforce them. Sufficient surety must be required from the contractor, and the question at once arises: What amount of surety is sufficient? It is a well-known fact, well-known, at least, to contractors, that the cost of maintaining a pavement under conditions to which it may be subjected for ten or even for five years may vary from 5 to 100% of its original

cost. The cases are quite frequent where the cost of maintenance will equal 50% of the original cost, even where the work has been faithfully and skilfully done. In those cases where the work has been done by an unskilful or dishonest contractor it is not safe to figure that less than one-half of the original cost may have to be expended to make good the guaranty for a period of even five years, particularly if the streets paved are subjected to heavy travel. Consequently, it may be stated as a general proposition that the surety required from contractors, in the case of asphalt pavements, will not insure the city against loss unless it is equal in amount to one-half the cost of construction. It will at once be replied to this statement that enormous areas of asphalt pavement have been constructed in American cities under guaranties running from five to thirty years, where the amount of surety required has varied from only 10 to 30% of the contract price, and that defaults by the contractor have been rare and unimportant. But it must be remembered that the asphalt pavements of this country have been constructed under a peculiar set of conditions. The work has been done largely by a few large companies whose business interest in the promotion and extension of asphalt pavements was such that they could not afford to default on these guaranties, even if their business responsibility and honor had not impelled them to comply with obligations incurred. Mr. Whinery.

There have been a great many cases in every large city having a considerable area of asphalt pavements where the surety in possession ofthe city has been less than one-third of the amount expended by the contractor to maintain the guaranty. There are many contracts involving a fifteen-year guaranty in existence in the City of New York, where the tangible surety in possession of the city does not equal one-half the amount it will cost the contractor to comply with the guaranty. It will depend entirely upon the honor and integrity of the original contractors or their successors, whether or not these contracts are lived up to until the end of the guaranty period. Sufficient examples have occurred to show what may be expected in cases where an irresponsible or dishonest contractor has been awarded work. The history of the asphalt pavement on Eighth Avenue, New York City, is a good illustration. How long the conditions prevailing in the past will continue, no one can say, but it may be safely predicted that, if the time ever comes when competition is as open in the asphalt-paving business as in other work, and when cities shall award contracts to the lowest bidder regardless of other considerations, it will be found necessary to require an amount of surety equal to at least half the original contract price.

There are two usual methods of providing surety for the integrity of such guaranties. In the one, a part of the money that would otherwise be due the contractor upon the completion of his work, is

Mr. Whinery. withheld from him until the expiration of the guaranty period. Now, if it be true that surety to the amount of one-half the contract price must be provided in order to make the city safe, the contractor would receive, under this plan, only 50% of the value of the work at his contract price. Knowing this, he must do one of two things. He may bid prices that will be sufficient, when 50% is retained, to pay the actual outlay for the work, or he may, if he possesses the capital or can command it, bid prices that will enable him to carry the retained balance during the period of the guaranty, as well as to pay the expenses of sustaining the guaranty. In the first case the work will be open to contractors of moderate or large means, alike, but the city must pay double the actual cost of the work. In the second case the contractor of moderate means will be shut out entirely, and a very few, of the wealthiest only, will be able to compete, and even then these wealthy contractors must bid prices that will compensate them for carrying the unpaid balance over a long period. In either case the city must pay the cost of the surety it requires. And, since contractors will usually bid prices that they think are on the safe side, particularly where conditions are such that exact cost cannot be determined, the city must usually pay a very high price for its security.

In lieu of retaining a part of the contract price, many cities require the contractor to give a bond with sureties deemed sufficient to make the bond good. These sureties may be either private persons or surety companies. Personal surety is notoriously unreliable. Even if the persons signing a bond are perfectly responsible at the time, they may be paupers before the end of five years. Whatever may be the cause or causes, it is a notorious fact that not in one case out of twenty, in our American cities, where the bond is forfeited is recovery actually made from personal sureties. It results that most cities now require contractor's bonds to be under-written by surety companies in good standing. But these surety companies are naturally averse to signing the bonds, of even the most responsible contractors, where they extend over a considerable period of years, and if they consent to do so they charge high rates for the service, and usually require collateral security from the contractor. The contractor must meet the expenses thus incurred, and must bid prices that will cover them, and the city must in the end pay liberally for the surety it exacts.

Without any intention of impugning the honor or the integrity of surety companies, it must be remembered that, corporations being without souls, they may possibly avail themselves of legal technicalities to escape the payment of large sums of money for which they are bound. Therefore, it is exceedingly important that no possible grounds of invalidity exist in contracts thus secured. In very few instances, the speaker believes, have surety companies been called

upon to make good for defaulting paving contractors; but it is not impossible that, if called upon to pay large sums on account of such default, it would be discovered that even this class of surety is not always to be relied upon implicitly. Mr. Whinery.

It has been claimed that contractors, in framing their bids, do not add much, if anything, to their prices to cover the expenses of a guaranty, but such a claim is too absurd to be considered seriously. The contractor, usually, is not a blockhead, and he is not in the business simply for benevolence or amusement. It is true, he cannot estimate exactly what it will cost him to maintain the guaranty, and if he is new to the business he may very greatly under-estimate that cost, but, whatever he thinks it may be, he adds it to the price he would otherwise bid, unless, indeed, he counts upon repudiating or escaping the guaranty entirely.

It may be that sharp competition, and a desire to control the business, will cause him to bid prices which he knows will not cover the cost of the work, but, if he has had any experience in the business, he knows that the cost of the guaranty is as palpable a quantity as is that of the pavement itself.

It must be evident, therefore, that the city must pay very liberally for the benefit the guaranty is supposed to confer.

In the second place, the question of the legality of long-time guaranties demands more consideration than it has received. Where pavements are paid for by special assessments upon the property benefited by the improvement, the courts have held, almost universally, that while the original cost of the improvement may be assessed against benefited property, the subsequent cost of maintaining the work cannot be thus assessed. In those decisions which uphold the validity of time guaranties, this principle is not denied, but an attempt is made to evade it. They set up the plea that the terms of the guaranty do not necessarily require the maintenance, in the proper sense of the word, of the pavement, but only that the contractors shall do the work with such materials and such skill that maintenance will not be necessary during the guaranty period; and if it shall become necessary to repair the pavement within that period, the fact is simply evidence that the contractor did not do the work in the manner required, and, therefore, must make good the consequences of his failure. This reasoning, in the speaker's opinion, is specious and erroneous. It may be sound, as an abstract legal theory, but, when confronted by the facts as they are known to every practical man, its sophistry cannot but be apparent. It may be safely asserted that no street pavement, subjected to even moderately heavy travel, will endure for five years, much less for a longer period, without the necessity for more or less repair, which no fair-minded person competent to judge can attribute to defective construction. The use of

Mr. Whinery. the familiar phrase, "ordinary wear and tear excepted," in very many guaranty clauses tacitly admits the fact. It is obvious that the repairs necessary to a pavement after it is two or three years old may be divided into two classes: First, those that may be due to the use of materials or labor not up to the requirements of the specifications; and, second, those made necessary by the wear and tear of use, whatever may have been the character of the original construction. Under the general principle of law referred to above, the contractor, in the first case, may be clearly held to his guaranty without danger of legal complications. Under the second, the property owner may justly object to being assessed specially for what is clearly maintenance of the work.

The decision of the Alabama Court, cited by the author, it seems to the speaker, is a striking example of fine-spun legal theory misapplied to practical facts and conditions. The learned judge concludes his decision with a fine-sounding dictum which was doubtless intended as a clinching argument in the legal knock-down of the whole fabric of the theory opposed to that held by the Court. Speaking of the requirements of the guaranty clauses in question, he says: "It is an incident of the contract, not an independent undertaking."

When, it may be asked by the practical man devoid of legal acumen, does such a guaranty clause cease to become an incident of the contract and not an independent undertaking? If it is an incident until the end of five years, why not until the end of ten years? And why may it not continue to be an incident during the whole life of the pavement?

If it is answered that it continues to be an incident only as long as may be necessary to establish the fact that the work was done with such material and skill as the contract called for, other questions arise. Just what length of time is required to disclose defective workmanship and materials? Is that period of time, assuming that it can be determined, a fixed and well differentiated period, or may it be affected by conditions which must vary with different streets, and even with different parts of the same street? Is the "incident" period the same on such a street as Broadway, New York City, and on the residence streets of the smaller cities, or even of those of New York City? Is it true that five years' use of a pavement on a heavily traveled street in any large city will develop nothing more than inherent defects in the original construction? And, that the effect of wear and tear of travel, which is not an incident of the contract, will begin the day after the expiration of the five years and not before? And, if it does not begin on the last day of the five years, when will it begin? If the guaranty of the pavement and the expense of maintenance it entails is merely an incident of the contract in the sense the Court seems to hold, would the contractor add anything to his price because of the incident?

Mr. Whinery.

If it is found, as a matter of fact, that the expense of maintenance entailed by the guaranty is invariably added by the contractor to his estimate of first cost, or is always considered in fixing the price in his original bid, does the Court still hold that it is merely an incident of the contract?

Notwithstanding the great importance of the question of the validity of these long guaranties, decisions of the higher Courts covering it have been comparatively few, but a large majority of these have been to the effect that they are not legal when they relate to work paid for by special assessment. Cases involving the legality of these guaranties apparently have not been brought before the Courts in a large majority of the States, and in these it remains an open question. The far-reaching effects and the serious financial results which would attend adverse decisions in many of the older States, where hundreds of thousands of square yards of asphalt pavement are covered by such guaranties, make the question one of very serious importance.

The absence of litigation and resulting decisions in so many of the States may be largely due to the fact that the large companies, by whom the greater part of the asphalt pavement in this country has been constructed, have, for reasons of their own, consented to, if they have not actually encouraged, the requirement of, long-time guaranties.

There is one important feature of these long-time guaranties that seems not to have been brought before, or considered by, the Courts at all; that is, that in nearly every city the terms of contracts for paving, including the guaranty requirements, are general, and are made to apply to all streets alike. No allowance is made with respect to different streets for different conditions of use. Thus *A* and *B*, two parallel and contiguous streets, are paved at the same time, under the same form of contract, and possibly by the same contractor. *A* is a main business street with very heavy travel, while *B* is a residential street, with very little travel. The work on both may be done with the same materials and with the same degree of care and skill, so that the pavements when completed are practically identical. The contractor, in framing his bids, having the facts in mind, would almost certainly bid a higher price per square yard on *A* than on *B*, because the maintenance of the pavement for the guaranty period will cost very much more on *A* than on *B*. Let it be assumed that the period of guaranty on *B* be such that its pavement would just endure to the end of the period without any repairs due to maintenance proper. During the same period, the pavement on *A*, subjected to very heavy travel, will have required a large expenditure for repairs due to the wear and tear of use. To pay for the pavements, the property owner on *A*, obviously, must be assessed for a much larger sum per unit than the property owner on *B*. If the property owner on *A* should resist

Mr. Whinery the assessment on the ground that a part of his assessment was for maintaining the pavement and not for construction, and should appeal to the Courts, it is difficult to understand upon what ground his plea could be refused, or the validity of the guaranty sustained. This particular ground of invalidity could be avoided by such a change in the nature of the guaranty as would require that the pavement should endure a stipulated, definite quantity of use, without showing indications of failure, as, for instance, the passage over it of a certain number of tons of travel, the quantity to be ascertained by censuses of travel taken at intervals of time in accordance with specified rules and regulations. The speaker has long believed that this is quite practicable, and that it constitutes the only rational and just basis for guaranties that are intended to test the endurance of a pavement.

At this point may be considered some provisions of these guaranties which have their foundation in the attempt to keep their requirements within legal bounds.

If the contractor is required to maintain the integrity of a pavement for a period of years, care must be taken that nothing is done by the city, or by other persons with the permission of the city, that will release the contractor from his obligation. For instance, the city may not remove and then repair a part of the guaranteed pavement, nor may it authorize persons other than the contractor to do so, since not only may the contractor claim that the adjoining pavement was injured in the operation, but he may claim that defects appearing later are within the area of the pavement disturbed and repaired by parties other than himself, and for which he cannot, therefore, be held responsible. It is generally difficult, if not impossible, even with the aid of carefully prepared diagrams, to locate accurately, after the lapse of a year or two, the exact boundaries of such repaired areas. Therefore, it has been found desirable, if not essential, to couple with the guaranty a provision that the guarantor shall make all necessary cuts into and replacements of the pavement, at a stipulated price. Usually, not much consideration has been given to the reasonableness of this price, as it has been assumed that it was an unimportant item. Very commonly, the price has been fixed at a certain percentum above the contract price for the construction of the pavement. If, therefore, the guaranty period was long and the contract price accordingly high, the price for such repairs was quite likely to be exorbitant. The result is illustrated nowhere better than in New York City, where, owing to extensive improvements in transportation systems and other underground structures, enormous areas of pavement have to be removed and replaced at prices so high as to make the cost of the work a very serious burden to those who must pay for it. It is probably true that this repair work is to-day, in New York City, the principal source of profit to the contractors who laid the pavement, or to their

successors. Even if the price for repairs, thus stipulated, were reasonable at the time the contracts were made, the great decline in the cost of asphalt pavement since that date makes them now abnormally high, and suggests the conclusion that it is unwise to continue a practice which seems necessarily to involve the making of contracts extending over long periods of time for supplies or services, the market value of which is likely to be subject to great fluctuations. Mr. Whinery.

Before dismissing this branch of the subject, it is worth noting that, where maintenance is embraced in contracts for construction, the cost of maintenance being embraced in the price bid, the contractor practically receives payment for the maintenance in advance, and, in the cases of long-period guaranties, so many years in advance that the resulting loss to the city is very great. In some of the contracts for the pavement of Broadway, New York City, it is very evident that the contractor, in fixing the price bid by him, regarded the maintenance as likely to cost nearly twice as much as the construction. Yet the city paid him the whole sum upon the completion of the work of construction. It is true that a large percentage of the sum was retained by the city, but this was held as surety, additional to the bond given, for the performance of his obligations (and it may be said, in passing, that as security it was very inadequate), and was not withheld with the view of paying for a part of the service when that service should be rendered. When it is considered that the largest part of the cost of maintenance will almost certainly be expended in the last half of the guaranty period, it will be appreciated that the advance payment must be a matter of large gain to the contractor and of large loss to the city.

There is another point which is of sufficient importance to merit consideration in this connection. If the city requires the contractor to guarantee certain results, he may justly claim that he must be left free as to the means by which these results are to be obtained, and that if the city assumes to dictate, as by prescribing definite specifications and the means to be used, he, the contractor, cannot be held responsible for results. It would seem, therefore, a very dangerous procedure, from a legal point of view, for a city to attempt to compel a contractor to guarantee work, constructed not in accordance with his judgment and experience, but in strict compliance with specifications formulated by the city's agents.

In the early history of asphalt pavement, when its merits were questioned and distrusted, when the properties of the material used were not well understood and the methods of constructing it were not familiar to engineers, it cannot be doubted that the requirement of some guaranty of results was not only justified, but was required by ordinary business prudence. But that time is now past. The essential quality of the material is known and can be determined in the

Mr. Whinery. laboratory or by experiment, and the principles of its construction are well understood. The fact that engineers do not now hesitate to prescribe how the work shall be done, down to its minutest details, is sufficient evidence that they feel competent to deal with the subject as freely as with any other engineering problem.

It is scarcely creditable to the profession that in the present state of the art and science of pavement construction we must depend upon contractors' guaranties to secure good work.

The necessity for a guaranty of endurance, therefore, seems no longer to exist. In view of these conditions, and others which time does not permit the speaker to detail now, he is, and has been for many years, of the opinion that it is both unnecessary and unwise to continue longer these long-time, or, as they may more properly be called, endurance, guaranties. He does not believe that, upon the whole, they result to the advantage of the cities, or are worth what they cost. There seems to him to be ample evidence that, whether judged from the economical or the legal standpoint, it is unwise as well as dangerous to complicate construction contracts with provisions for maintenance in such a way that they are not readily separable. A guaranty extending over a short period, sufficient to disclose defects of construction that might have been overlooked as the work progressed, would be entirely unobjectionable. It should not extend over two years. With such a guaranty our cities can safely rely upon their engineers to prepare adequate specifications, and to enforce them so as to secure work of the highest standard of excellence. In cases where the local engineering talent may happen to be without the requisite knowledge and experience, there are not a few specialists whose services could be readily secured, in the capacity of consulting engineer, as is the practice in other branches of engineering work.

If it is thought advisable, maintenance of the pavement, after the expiration of the short guaranty, can be made by contract either with the contractor who constructs the pavement or with others.

Some of the advantages claimed for the long-period guaranty could be secured by embracing in the original contract for construction provisions for the maintenance of the pavement, after the expiration of the short guaranty, at a stipulated price per square yard per year, payable yearly when the service shall have been rendered. In case the work is to be wholly or partly paid for by special assessments, only the cost of construction could then be assessed upon property owners, and the cost of maintenance could be paid from the general funds of the city.

The speaker agrees with Mr. Lewis that the plan adopted by the author is in the right direction, if long-period guaranties are to be continued, as it attempts to separate the guaranty from maintenance. But the plan adopted involves, to some extent, the common fallacy

that the cost of guaranteeing the work on one street is practically the same as on any other street. Thus, the author bases his price for maintenance on the average cost of maintenance in Washington and other cities, ignoring the fact that the conditions in Mobile may be such as to make such average cost totally inapplicable to the streets of that city. If his plan were adopted in New York City under general specifications, used, as is customary, for the whole season's work, the absurdity of applying the Washington average, or of assuming that the specified maintenance price would apply alike to Broadway and to a short residential street in the Borough of the Bronx, would be palpable. If the author had carried his plan a little further and had asked bidders to name prices for maintenance during the second and third periods of five years, as well as a price for construction, he would then have approached a rational method; but even then he would have compelled the contractor to guess at the conditions that would probably prevail on a certain street fifteen years in the future. The speaker uses the word "guess" purposely, because, in our comparatively young and rapidly developing American cities, no one can foretell what changes in the growth and distribution of travel on any street may take place within a period of even ten years. Mr. Whinery.

Mr. Lewis evidently has the impression, which is thought to be quite general, that the contractor, in making up his bid, adds little or nothing, to his price for construction, to cover the guaranty.

It was a part of the duty of the speaker, while connected for many years with one of the asphalt paving companies, to prepare hundreds of bids every year, and he may say that, in estimating the cost of the work, the cost of maintaining the guaranty was, in every case, considered and incorporated. The cost thus arrived at was not, however, always the price bid, because various circumstances, and the exigencies of competition, often dictated the price bid, regardless, within certain limits, of the estimated cost.

Nelson P. Lewis, M. Am. Soc. C. E.—The author has the object of trying to eliminate the necessity of a contractor making a guess at the cost of maintaining a pavement which may be laid by him during the period for which such maintenance will be required. His object is certainly a commendable one, and the speaker believes he is moving in the right direction. Mr. Lewis.

Let us see just how his specifications work out. They provide that all repairs which may be ordered during the period of the contractor's guaranty will be paid for, if the area is less than 20 sq. yds., at a price 20% greater than his original contract price, while for areas between 20 and 50 sq. yds. the advance will be 15%, and for areas over 50 sq. yds. 10% over the contract price. He refers to contracts let in Mobile for 17 000 yds. of pavement at $1.15 per square yard. The amount paid for repairs, therefore, will be $1.38 per square yard, if the area be less

Mr. Lewis. than 20 yds. If more than 20 yds. and less than 50 yds., \$1.32¼; while for any repairs covering more than 50 yds., he will be paid \$1.26½ per square yard.

Now, he assumes as a proper maintenance cost to the city for the first five-year period after the expiration of a five-year guaranty, or for the second five years of the life of the pavement, 3 cents per square yard per annum. Of the 17 000 sq. yds. laid in Mobile—assuming that practically all the repairs are in areas of less than 20 sq. yds., as will, in the speaker's opinion, be the case in a successful pavement—he would be expected to relay about 370 yds. annually during the first five years, or 2.2% of the total area each year. For the second five years, during which he is allowed 8 cents per square yard, he could relay just about 1 000 sq. yds., or 6% of the total area each year. Now, if he assumes that none of these repairs overlaps another, which is not probable, he would have renewed during those ten years, from the fifth to the fifteenth year, 41% of the original surface. Of course, there will be more or less overlapping; repairs will wear out and have to be renewed, and it is fair to assume that, say, one-fifth of the repairs made will have to be done a second time, so that the contractor will have replaced one-third of the total area of the pavement by the end of fifteen years.

Has the author adopted a fair standard when he refers to the cost in the City of Washington? The speaker does not believe he has. Washington is not a logical city, to which one may look for such a standard, especially if it is to be applied to a small city which is just beginning to lay smooth pavements. The conditions in Washington are nearly ideal. The streets are extremely broad, the traffic of the city is comparatively light, and it is admirably distributed. In smaller cities, when smooth pavements are first laid, an abnormal amount of traffic is always attracted to them. This is invariably the case, and it is impossible to predict what will be the relative amount of traffic on new, smooth pavements. The character of a street will be radically changed within two or three years, possibly, after it is paved. Even in a city having comparatively a large amount of smooth pavements, a remarkable transformation will take place in the vehicular traffic as new routes are laid out and new connections are established.

While the author may have given the contractor a good guessing basis, he is still going to make him guess. He is compelled to guess how much he will have to add to his construction price in addition to the 3 cents per square yard allowed for the second five-year period, and the 8 cents per square yard for the third five-year period, or, in other words, how much more than 40% of the original surface of the pavement will have to be renewed by the end of fifteen years, and he is going to guess liberally if he is a shrewd man. The contractor in Mobile evidently did not make a very liberal allowance, as his price was only \$1.15 per square yard.

It is evident from the specifications which are quoted in the paper that if the contractor does not have to lay as much as was anticipated, that is, if he does not have to renew 2.2% annually for the first five years and 6% annually for the second five years, he is not to be paid for it. Mr. Lewis.

Now, there has been a good deal of what the speaker will venture to call nonsense said and written about long-time guaranties and short-time guaranties, and the reasons for one or the other.

In a report made to the Mayor of New York City by his Commissioners of Accounts in 1899, there was a table tending to show what the insurance cost of pavements had been, that is, what premium the city was paying for long-time guaranties, and in estimating that cost they allowed 2 cents a square yard annually for the maintenance of New York pavements, amounting to 30 cents per square yard for the entire fifteen years, which was the guaranty period in the contracts under discussion. If this sum were all charged to the ten years from the fifth to the fifteenth, it would amount to only 3 cents per square yard per annum, and it cannot be claimed seriously that New York pavements can be maintained for 3 cents per square yard between the fifth and fifteenth years. The table also includes the cost of a plant for fifteen years at 5 cents per square yard annually, and the last and largest item is interest lost on money retained for fifteen years at 6%, amounting to 64½ cents, showing that the total cost to the city for insuring its pavements for fifteen years is 99½ cents per square yard.

It is difficult to understand how it can be claimed seriously that the money retained under a paving contract, as it is in New York City, is wrongfully kept from the contractor, and that he should be allowed and will charge interest on it. It is not money which has been earned.

Mr. Whinery has referred to cases where 50% of the amount due the contractor has been held back that should have been paid to him on the completion of his work. It is probable that he did not refer to the New York City practice, but to cases where no bond is required.

The amount retained by the City of New York under the fifteen years' guaranty was 30% of the cost of the asphalt surface, of which 3% was paid each year, beginning at the end of the sixth year and up to the end of the fifteenth; that is, it was assumed that 3% of the cost of the pavement, which, at $2 a yard, would be 6 cents, or, at $3 a yard, 9 cents, was the amount which the contractor would probably earn in making those repairs, and this seems to the speaker a fair allowance. In the case of the ten-year guaranty, the amount retained was 20% of the cost of the wearing surface, of which 4% was paid at the end of each year from the sixth to the tenth. This allowance again would be, on the basis of $2 and $3 pavements, 8 or 12 cents per square yard. But that money was not earned; it was not due the contractor, and to charge it up as the price the city was paying for long-term guaranties was erroneous and misleading.

Mr. Lewis. The speaker, however, agrees with the conclusions reached, and the contention that the long-term guaranty is unwise, but his reasons are entirely and radically different. It is probable that an asphalt-paving contractor, who is jealous of his reputation, will charge little, if any, less for a pavement without a guaranty than if he were guaranteeing it for five years, assuming that he would exercise the same care in laying the one as the other.

Streets differ so much in the traffic they sustain, and, as already stated, the future of a street is so uncertain, that when one deals in futures of more than five years he certainly has to estimate liberally, and the contractor is obliged to guess, and guess liberally, how much he will have to spend from the fifth to the fifteenth year.

But conditions are changing in a large city like New York, and they are changing for the better, and account should be taken of the changes. Steel tires are wider than they were, a large proportion of them has been replaced by those of rubber, horse-shoes are lighter, and one will not discount the future properly if he does not take into account the large percentage of horseless, smooth-wheeled vehicles which will be found on our streets before a ten or a fifteen-year contract made at the present time will have expired. All these changes will tend to simplify the problem of maintaining our asphalt pavements. The elements most injurious to them are constantly being eliminated.

Now, is it not the plain duty of the municipality to take advantage of these changed conditions, instead of giving all that advantage to the contractor? It will be his if he is obliged to make a guess upon the cost of maintaining a pavement for fifteen years. He will base his guess upon his past experience with pavements now ten and fifteen years old, and it can scarcely be questioned that the conditions will be so different in another ten or fifteen years that the advantage will all be his.

The speaker would like to suggest what seems to him a logical method of dealing with this problem, and one which will eliminate it almost entirely. It is that, after the expiration of the original contract covering not more than five years, subsequent contracts for from one to five years be made for maintaining the pavements, which contracts should provide that the contractor be paid for the material actually used, and which would be placed where directed.

This system has been in vogue in the District of Columbia for some years, and with admirable results. It is doubtless true that the figures which the author uses, viz., 3 cents per square yard as the cost of maintenance for the second five-year period and 8 cents per square yard for the third five-year period, are due largely to the use of this method, although, if the speaker correctly reads the recent reports of the District of Columbia, the average cost from the fifth to the tenth year is below 3 cents, and from the tenth to the fifteenth year below

8 cents. It is probable that the 8-cent average is obtained by including, as the reports do, a number of streets which are entirely re-surfaced and which increase the apparent cost very materially. Mr. Lewis.

This same method is now in use in the Borough of Brooklyn, the second contract made on this basis being now in force. Under these contracts the wearing surface is paid for by the cubic foot, measured in carts on the street. The first contract provided for a payment of 97 cents per cubic foot for the wearing surface; the present price is 90 cents a cubic foot. With the burner method, or the skimming process, the price under the old contract was $1.15 per cubic foot; and under the present one this method of repair is not provided for at all, it being considered unsatisfactory and to be discouraged. Under the first contract the price for binder was 35 cents a cubic foot, and under the present one 50 cents.

A great element of saving under such a contract is in repairs to streets, which, from their age or traffic, have been reduced in thickness. Many streets, doubtless, will be found in which the wearing surface, instead of being 2 ins., is 1¼ ins., and in some instances less. Formerly, when repairs were paid for by the square yard, as is provided for under the plan proposed by the author, the city would pay for a standard pavement 2 ins. in thickness, and with 1 in. of binder, when it gets considerably less. But, by the cubic-foot method, the same amount will lay a proportionately greater area, and while the amount expended under the Brooklyn contract was perhaps as much as under the old method, still the repairs were put where they did the most good. It enables one also, in the case of extremely bad pavements which need re-surfacing, to repair only the dangerous places and keep them in safe condition for a year or so, until they can be re-surfaced, without spending a large sum.

As already stated, the conditions in Washington are exceptionally favorable, but recent reports from Rochester would indicate that the repairs to its asphalt pavements cost even less than in Washington. The record, as given in the report of the City Engineer for 1900, is quite remarkable. The average cost for all streets is not given, but taking a pavement which was laid in 1886: From 1896 to 1900 the cost of maintenance per square yard per annum was 1.2 cents; another one cost 6.6 cents, and the next one 4 cents. These are the averages from the tenth to the fourteenth year. The next one is 2.8 cents, and one, on which the guaranty expired in 1892, appears to have been maintained since that date at an average cost of less than 1 cent per square yard. This is an admirable showing, and, judging from the limited opportunities the speaker has had to observe the Rochester streets, it is not because the repairs have been made spasmodically and only when necessary, because the streets have been kept in excellent condition. The City Engineer also states that in making the figures the area between rail-

Mr. Lewis. road tracks and 2 ft. outside was excluded from the area of pavements maintained and in the cost, as these spaces are cared for by the surface railroad companies.

In reply to the question as to the reason for the cracking of asphalt pavements subjected to little or no traffic, the speaker believes that such pavements do have a tendency to crack more than those which are subjected to moderately heavy traffic; the reason, undoubtedly, is that in unused or little used pavements the wearing surface is not kept thoroughly compressed, and is therefore much less dense, and the action of the elements on the bitumen, the function of which is to hold the mineral matter together, is quite marked. Traffic does keep the surface more dense, and therefore more impervious to water, and better protected against the action of frosts.

Mr. Thompson. S. C. Thompson, M. Am. Soc. C. E.—In considering the question of maintenance of asphalt, the speaker desires to call attention to some conditions which obtain in the Borough of the Bronx. Quite a number of strips for the use of bicyclists have been constructed in the roadways of the different streets and avenues. They are usually from 3 to 5 ft. wide, or of such a width as to prevent both wheels of a vehicle getting on them.

In most cases these strips have been laid on the old block pavement as a foundation. The blocks have been lowered and the asphalt wearing surface placed on the top, and brought up to the same cross-section as the remainder of the roadway. Most of these strips have been placed next to the curb. The result, in one street in particular, has been that the maintenance has been very expensive for the contractor, as the block pavement in the gutters, under the asphalt, allowed the water from the crown of the street to get under the strips, and, by its disintegrating influence and through the effects of frost, quite a large portion of the strip has been destroyed. This has made its renewal necessary at least twice during the past five years, and the speaker's belief is that it was due almost entirely to the fact that the foundation was unsuitable. The water draining from the crown of the street and getting under the asphalt caused it to crumble, so that it would not hold together, and the rolling received from the traffic was not sufficient to keep it in proper condition.

This calls attention to another matter, in the same line, *viz.*, paving with asphalt upon old blocks in general. The speaker desires to go on record as being opposed to the laying of asphalt on old block pavement, as done at the present time in New York City. The condition that exists is as follows: A street on which traffic has pounded the blocks down securely and solidly to a firm and unyielding foundation is to be repaved. The first thing that is done is to remove the blocks, excavate some of the earth, replace it with a sand bed, and relay the block pavement on the lower grade, thus getting the foundation into

the very condition that is not wanted, *viz.*, it is not thoroughly compacted and is easily moved. After leaving the street open to traffic for longer or shorter periods, varying according to the conditions that exist and the demands of traffic, the asphalt is laid upon these blocks. Mr. Thompson.

Having had a good opportunity to observe work done in this manner, it is the speaker's opinion that satisfactory results cannot be obtained thereby. If the asphalt could be laid directly upon the old blocks, which have been pounded down thoroughly by the traffic, good results would be obtained, but, if openings are to be made in the roadway after laying, these results will not be as satisfactory as when a uniform foundation, like concrete, is laid.

J. M. Evans, Assoc. M. Am. Soc. C. E.—The criticism has been made that an asphalt pavement must be subjected to a rolling process, similar to vehicular traffic, in order to insure it remaining in place; and, to substantiate this claim, certain people point, with more or less justification, to the walks in our parks and other places, such as plazas, footbridge floors, etc., where the asphalt is badly cracked, or has been displaced by frost. Mr. Evans.

The speaker would like to enquire whether this is a fact, or whether it is the fault of the foundation or the quality of the material used in the surface; also, if asphalt walks, as laid at the present time, are as durable as the concrete walks of the so-called granolithic and kindred variety?

As granolithic pavement is now put down, with joints to allow expansion, it has proved very efficient in New York and other cities. The claim is made for it that, under light showers, it does not present as greasy and slippery a surface as asphalt, and that it can be laid without an expensive plant. Is it practicable to lay asphalt with joints to allow contraction, and thus prevent cracking; and, if so, will the rolling traffic still be necessary to preserve the surface intact?

W. Boardman Reed, M. Am. Soc. C. E.—The speaker is not an expert on asphalt pavement, but has had considerable experience in the maintenance of such pavements beside street-railway tracks. In New York City, for several years past, it has been the practice to use what is known as the "Trilby" rail, having an inside flange, about $2\frac{1}{2}$ ins. wide, at the same height as the head, and to lay asphalt up against both sides of this rail. Where the rail is perfectly rigid, pavement of this character will last fairly well on streets with little traffic; but, where the traffic is heavy, there is an inclination for truckmen to follow the rail, and, dropping off it from time to time, wear ruts beside it. Mr. Reed.

On track which is not perfectly rigid it is almost impossible to maintain asphalt pavement next the rail in safe condition, and where

Mr. Reed. this has been attempted it has been found to be very expensive to maintain it even in passable condition. On one of the street-railway lines in charge of the speaker the cost of the maintenance of asphalt, between the rails only, has been $4 500 per year per mile of single track, or about 52 cents per square yard. Toothing stones have been laid next to the rail, and a cast-iron strip has been used with considerable success for this purpose, with the idea of allowing the rail to vibrate against either the toothing stone or the iron strip without injuring the asphalt or the foundation upon which it is laid.

Much of the difficulty in the maintenance of asphalt on streets comes from the frequent tearing up of the street for repairs to substructures. There is no doubt that asphalt pavement can be laid so as to last many years with very little repair, if a proper foundation is put in and if that foundation is not disturbed. This is especially noticeable on Fifth Avenue in New York City, but, even there, where excavations have been made, bad settlements of the pavement have occurred shortly after it has been restored.

One point in the matter of maintenance the author has not touched. A company laying asphalt under guaranty practically has control of the street during the existence of the guaranty, and would be able to interfere more or less with any improvements, even though they might be desired by the city authorities. Such a company is able to demand almost any price, from any corporation that makes such improvements, for the privilege of tearing up and restoring the pavement, if such improvements change the surface of the street in any manner. In one instance, in New York City, where the pavement had been laid under a five-year guaranty, there was in the street a single street-railway track, constructed and operated for horse-cars. It was considered desirable, not only by the street-railway company, but by the city authorities, to change the location of the existing track and put in two tracks, making them according to the standard slotted construction of New York City, instead of the original horse-car construction. The asphalt company held that the laying of the second track and the change in the style of construction invalidated their guaranty contract, and the railway company was obliged to pay, and, rather than have litigation, did pay, a considerable sum to the asphalt company for the privilege of complying with the desires of the city authorities.

In the speaker's opinion, the exorbitant prices bid in the past for the laying and maintaining of asphalt, with the privilege of charging any corporation that may be obliged to disturb the pavement the contract price for its restoration, are unjust. Quite recently, it was necessary to make repairs to a street-railway track on a street where pavement had been laid, under a fifteen-year guaranty, at $4.86 per square yard. The guaranty would expire in 1908. The asphalt company, according to contract with the city, had the right to charge the full

contract price for the restoration of this asphalt; whereas, under the present prices, the work could be done for less than $2 per square yard. Mr. Reed.

With regard to laying and maintaining asphalt beside the rail; in the year 1900, a surface track was laid on 34th Street, New York City, with a 9-in. girder-rail supported by a heavy concrete beam. This rail was perfectly rigid, there being no vibration, even though very heavy cars were operated over it. Special care was taken in the laying of the asphalt, yet it was not more than six or eight months before ruts appeared beside the rail, and it was necessary to make repairs.

Asphalt wears out rapidly at any point where it adjoins a harder material, as for instance, at the manholes of sewers, at water-gate boxes, etc., owing to the pounding of vehicles dropping from the hard to the softer surface.

James N. Hazlehurst, M. Am. Soc. C. E. (by letter).—In exercising his privilege of closing this discussion and of replying to criticism made by participating members, while disclaiming any intention of being captious or hypercritical, the writer is somewhat uncertain of his position and attitude on account of the considerable amount of irrelevant matter interjected by participants. Thus, one of these gentlemen objects to laying asphalt strips upon old block pavement for the use of bicyclists; another seeks information as to the cause of the rolling and cracking of asphalt pavements; while, of the two members who attempt to discuss the paper seriously, the remarks of one, if not entirely irrelevant, are, at least, full of generalities, as, for instance, the necessity for and character of the bond intended to secure the maintenance clause of such a contract; the possible increase of price due to the subsequent care of the pavement but concealed in the original proposal; a reiteration of the principle (stated by the writer) of the shadowy and narrow division line between maintenance and guaranty clauses; a recitation of the well-established fact that, to preserve the integrity of a contract for maintenance, due care must be taken by the city that acts of omission or of commission do not release the principal or relieve the surety; followed by a statement of the experience of New York City in having to pay an exorbitant charge for repair work which, although included, was overlooked in the consideration of the original contract; and finally, as an original proposition, suggesting that the contractor be allowed to follow his own judgment as to mixtures and measures if it is expected to hold him to specific and definite results. Mr. Hazlehurst.

This general treatment of the subject, while interesting, brings out only one or two new points, and to these the writer desires to reply briefly. According to Mr. Whinery:

"There is one important feature of these long-time guaranties that seems not to have been brought before, or considered by, the Courts at all; that is, that in nearly every city the terms of contracts for paving, including the guaranty requirements, are general, and are made to apply to all streets alike. No allowance is made with respect to different streets for different conditions of use." etc.

Mr. Hazlehurst. While it is admitted that the character of the traffic upon the streets possibly included under such a maintenance clause is variable, the conditions are possibly not more eccentric than those surrounding the same streets, let at the same time, and under the same specifications for constructive work, and upon which the unit price bid is the same, in both cases, it being entirely possible for the contractor to balance his estimate of cost so as to reach a fair average price both for construction or for maintenance.

Again, Mr. Whinery declares that:

"In the early history of asphalt pavement, when its merits were questioned and distrusted, when the properties of the material used were not well understood and the methods of constructing it were not familiar to engineers, it cannot be doubted that the requirement of some guaranty of results was not only justified, but was required by ordinary business prudence. But that time is now past," etc.

While it is a fact beyond contention that certain well-known brands of asphalt are thoroughly understood and may be fully relied upon under given conditions, to discriminate in their favor produces a limited competition, to avoid which and yet to safeguard municipal authorities, it is proposed by the writer to rely more fully upon the guaranty of the contractor for successful and permanent work.

The writer can hardly believe that even as experienced an engineer as Mr. Whinery is conceded to be would be willing to rely entirely upon the determination by himself and his chemist of the particular merits of an untried asphaltic material, such as the bituminous rock or liquid product of California, Texas or Utah. If he contends that such unknown material should be barred, as has long been the habit of many of our larger cities, there is little encouragement offered to honest competition against the larger companies, and he would endorse and perpetuate monopoly, collusion and the whole train of ills due to restricted competition and class legislation.

Furthermore, notwithstanding Mr. Whinery's assertion, that "Our cities can safely rely upon their engineers to prepare adequate specifications, and to enforce them so as to secure work of the highest standard of excellence," the tendency is distinctly toward contracts with a longer maintenance or guaranty clause, and where, formerly, five years was considered a sufficient term of guaranty, ten years is now more common, and fifteen years is not an unusual requirement in this country, while the standard term for maintenance in German cities is nineteen years!

Mr. Nelson P. Lewis, in his consideration of the subject, calls into question the fairness of taking the recorded cost of maintenance for the asphalt pavements of the City of Washington as a representative and equitable basis of cost, but without pausing to justify the correctness and fairness of the reference figures, the writer's contention is for the principle or system of fixing a stipulated sum as the maximum

Mr. Hazlehurst.

price for the maintenance of the pavement, not to be paid as a bonus, but only as earned; and contends that such an arrangement furnishes a fair working hypothesis, although admitting that the contractor will have to make a "guess" as to the actual cost of such maintenance. Is there ever any certainty in the bid of a contractor for the execution of a contract? Then, why should it be less legitimate for the contractor to "guess" at the cost of maintaining any piece of work than for him to "guess" at its first cost?

Referring to Mr. Lewis' recommendation that:

"After the expiration of the original contract covering not more than five years, subsequent contracts for from one to five years be made for maintaining the pavements, which contracts should provide that the contractor be paid for the material actually used, and which would be placed where directed."

The advantage of this method of securing necessary material for repairs would be, as suggested by Mr. Lewis, the element of saving under such a contract, in the repairs to streets, which, from their age or traffic, have been reduced in thickness. This method of arriving at an equitable measurement of the value of the required materials for repair is novel and undoubtedly possesses good points, but the annual wear and tear of the street surface, and its consequent loss in thickness under traffic may be discounted by the contractor and equated for in the original proposal for construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 941.

IMPROVEMENT OF THE BLACK WARRIOR, WARRIOR AND TOMBIGBEE RIVERS, IN ALABAMA.*

By R. C. McCalla, M. Am. Soc. C. E.

With Discussion by Messrs. William L. Sibert, D. A. Watt, George Y. Wisner, John M. G. Watt, George T. Nelles, Edward P. North, S. Whinery, George W. Rafter, Theodore Belzner, William M. Hall, B. F. Thomas, D. M. Andrews, Nat. A. Yuille and R. C. McCalla.

General Characteristics.

The Black Warrior, Warrior, and Tombigbee Rivers, together with the Mobile River, form a chain of rivers flowing through the center of the great Warrior Coal Basin, and entering the Gulf of Mexico through Mobile Bay. The Black Warrior River is formed by the junction of the Mulberry and Locust Forks. At Tuscaloosa the name changes to Warrior. The Warrior flows into the Tombigbee about one mile above Demopolis, and the Tombigbee and Alabama Rivers join and form the Mobile River about 45 miles above the City of Mobile. Fig. 1 is a profile of these rivers, with the elevations of the locks. Fig. 2, a map of Alabama, shows the location of the rivers. The total length of these streams from the junction of Mulberry and

* Presented at the meeting of September 3d, 1902.

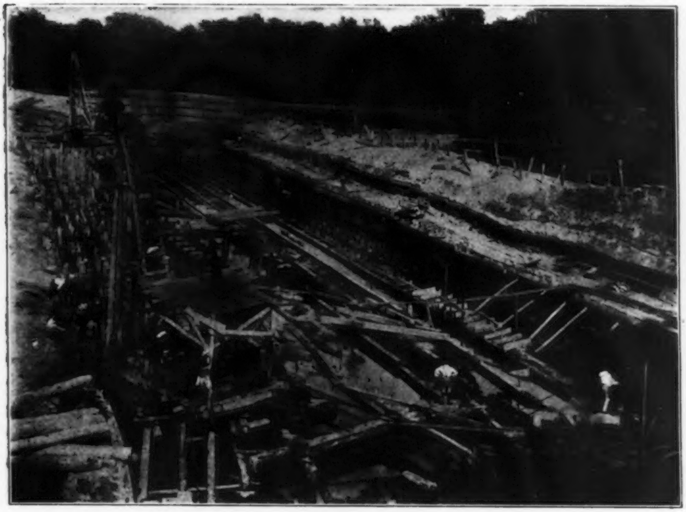
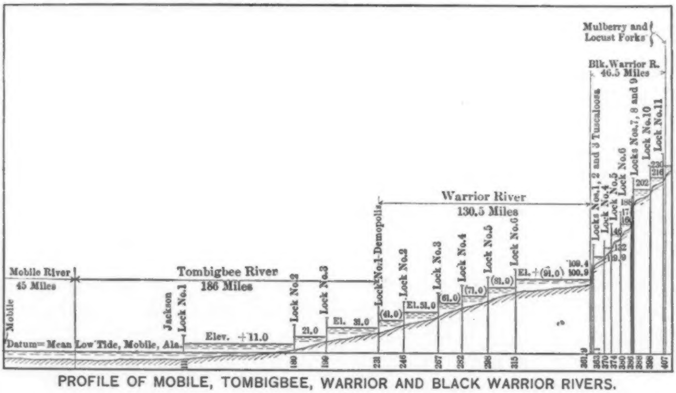


FIG. 1.—WARRIOR RIVER: SITE OF LOCK NO. 5, LOOKING UP STREAM

FIG. 2.—WARRIOR RIVER: LOCK NO. 6, SHOWING CONCRETE FORMING

Locust Forks to Mobile is 408 miles, and the total fall at low water is 214.5 ft., as shown in Table No. 1.

Above Tuscaloosa the Black Warrior flows through a narrow gorge enclosed by steep hills and perpendicular rock bluffs. At low water there is a succession of level pools connected at frequent intervals by steep, shallow, rock rapids varying in fall from a few inches to 19 ft. The bed of the stream is generally of hard sandstone. Below Tuscaloosa these streams flow through alluvial valleys several miles wide, and are subject to overflow during floods. The bed is generally composed of thick layers of sand and gravel underlaid with clay or soft limestone. Occasional ledges of the latter rise nearly to the low-water surface, and form reefs across the river, but most of the shoals have beds of sand and gravel, and there is rarely more than 1 ft. fall through one shoal.



PROFILE OF MOBILE, TOMBIGBEE, WARRIOR AND BLACK WARRIOR RIVERS.

FIG. 1.

The minimum discharge of the Black Warrior and Warrior Rivers is about 150 cu. ft. per second, and the maximum discharge about 150 000 cu. ft. per second. The minimum discharge of the Tombigbee River below Demopolis is about 400 cu. ft. per second, and the maximum discharge probably approaches 300 000 cu. ft. per second. Large floods usually occur during the winter and spring, and at rare intervals during the summer and fall. They carry vast quantities of drift wood, and sometimes cause considerable loss of property. Smaller rises, which interfere with construction, but do no other damage, usually occur several times during each low-water season, from June

to December. The most rapid rate of rise ever recorded at Tuscaloosa was 5 ft. per hour for 4 hours, after which the rate gradually decreased; but rises of 2 ft. per hour are not uncommon. The rate of rise is usually greatest at Tuscaloosa, and gradually decreases below that point. The maximum oscillation of the Black Warrior River varies from 15 to 50 ft. at different points. The maximum oscillation of the Warrior and Tombigbee Rivers is 67 ft. at Tuscaloosa; 47 ft. at Gray's Landing, 42 miles below Tuscaloosa; and 70 ft. at Demopolis, below which place it gradually decreases.

TABLE No. 1.

Description.	Length, in miles.	Fall, in feet.	Average low-water slope, in feet per mile.
Black Warrior River, Forks to Tuscaloosa......	46.5	128.5	2.76
Warrior River, Tuscaloosa to Mouth of Warrior........	130.5	58.5	0.45
Tombigbee River, Mouth of Warrior to Mouth of Tombigbee........	186.0	27.5	0.15
Mobile River, Mouth of Tombigbee to City of Mobile........	45.0	0.0	0.00
Totals and average........	408.0	214.5	0.53

Previous to Improvement.

The Mobile River is tidal, and has ample depth for river navigation without improvement. Previous to improvement the Tombigbee River was navigable for light-draft steamboats to Demopolis about 9 months per annum, and the Warrior River to Tuscaloosa about 4 months per annum. Tuscaloosa was considered the head of navigation. Rafts and flatboats were brought down the Black Warrior on floods, but there was no other navigation on this stream.

Early Improvement.

Prior to the beginning of work on the present project, in 1888, no work was done on the Black Warrior, and work on the Tombigbee and Warrior Rivers was confined to removing snags, cutting overhanging trees, dredging through shoals, and building light dikes and training-walls to confine the water to the channel. This work was of considerable benefit to navigation, but did not extend the boating season materially, or do away with the uncertainty of navigation during a part of each boating season.

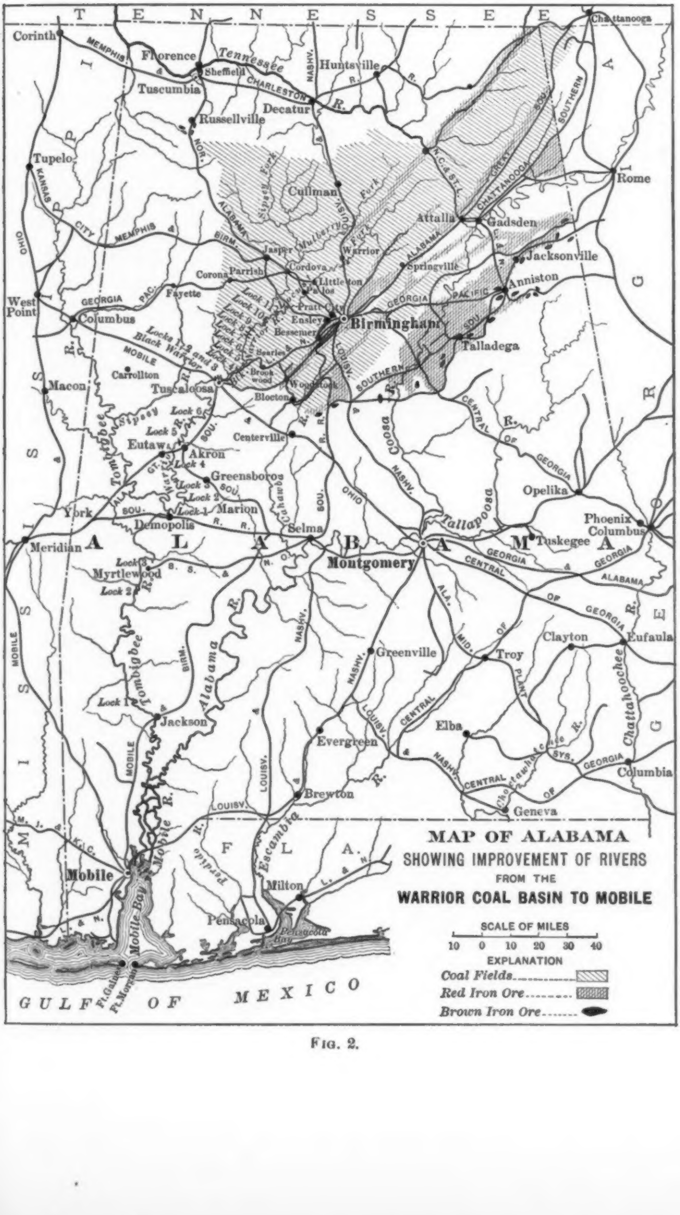

MAP OF ALABAMA
SHOWING IMPROVEMENT OF RIVERS
FROM THE
WARRIOR COAL BASIN TO MOBILE
SCALE OF MILES
10 0 10 20 30 40
EXPLANATION
Coal Fields
Red Iron Ore
Brown Iron Ore
TENNESSEE
GULF OF MEXICO
Birmingham
Montgomery
Mobile
Tuscaloosa
Selma
Anniston
Huntsville
Chattanooga

FIG. 2.

Present Project.

The present project is to obtain a waterway for the transportation of coal, in barges of 6 ft. draft, from the Warrior coal fields to the Gulf of Mexico, by a slack-water system of locks and dams. There will be twenty locks and dams, with a total lift of 230 ft., the upper lock and dam raising the low-water surface about 15.5 ft. at the junction of Mulberry and Locust Forks. The total cost will approximate $5 000 000. Beginning at the lower end, there will be: In the Tombigbee System, Lock No. 1 of 11 ft. lift and Locks Nos. 2 and 3, each of 10 ft. lift; in the Warrior System there will be Locks Nos. 1 to 6, each of 10 ft. lift; in the Black Warrior System there will be Lock No. 1 of 10 ft. lift, Lock No. 2 of 8½ ft. lift, Lock No. 3 of 10½ ft. lift, Lock No. 4 of 12 ft. lift, and Locks Nos. 5 to 11, each of 14 ft. lift. The elevations of these locks are shown on the profile, Fig. 1.

The locks are to have 52 ft. clear width, 286 ft. available length, 322 ft. length between hollow-quoins, and minimum depths of 6½ ft. on the miter sills and 7 ft. in the lock chambers. Steel lock gates and fixed dams about on line with the upper gates are to be used throughout. The work is now in progress, and is being carried out by the United States Government under the continuing contract system of river and harbor appropriations.

Work Accomplished—Tombigbee System.

Lock No. 1 is located and partly built at McGrew's Shoal, 111 miles above Mobile, on a reef of soft limestone which extends across the river a few feet below low water. Work on this lock was begun in 1896, and suspended for lack of funds in 1899. The work was done with hired labor, and was greatly retarded at various times by leaks in the coffer-dam, freshets and lack of funds. The lock walls are finished, but the gates and valves are not completed, and no work has been done on the dam or abutment. The lock walls are of concrete, composed of 1 part loose cement, 2½ parts sand, and 6 parts pebbles. They are founded directly on the soft limestone, trenches in which were dug about 2 ft. deep under each wall and filled with concrete for a footing course. Lagerdorfer Portland cement was used throughout. The dam and abutment will also, probably, be built of concrete.

The river below Lock No. 1 is tidal, and needs no improvement except dredging in places. Locks Nos. 2 and 3, between Lock No. 1

FIG. 1.—BLACK WARRIOR RIVER: SITE OF LOCK NO. 4, LOOKING DOWN STREAM.

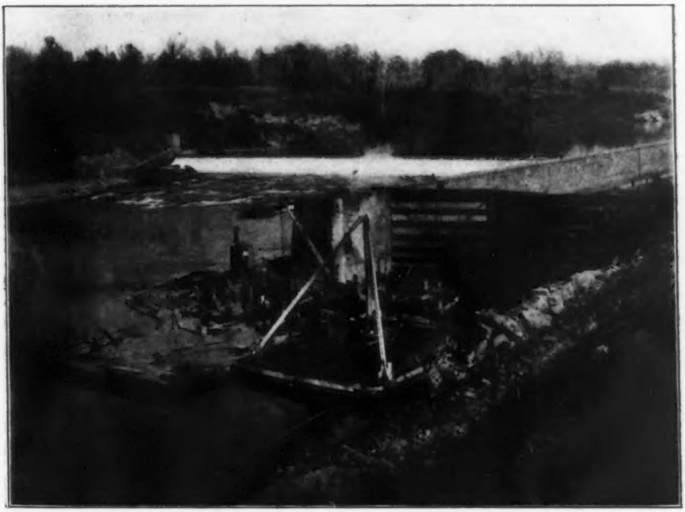


FIG. 2.—BLACK WARRIOR RIVER: LOCK AND DAM NO. 1, LOOKING UP STREAM.

and Demopolis, have not yet been located, and no appropriation has been made for their construction.

Work Accomplished—Warrior System.

The surveys, borings and descriptions of lands needed for Locks Nos. 1, 2 and 3, have been completed, but no appropriation has yet been made for their construction. The general features of their design will follow closely those for Locks Nos. 4, 5 and 6 of the same system. The locks and abutments will be of Portland cement concrete on pile foundations. The dams will be timber cribs on pile foundations, similar to those at Locks Nos. 4, 5 and 6, of the Warrior System. Lock No. 1 is located in the Tombigbee River, just above Demopolis and just below the mouth of the Warrior, 230.6 miles above Mobile. Locks Nos. 2 and 3 are located in the Warrior River, 246.1 and 266.7 miles, respectively, above Mobile.

Locks Nos. 4, 5 and 6 are 282.2, 298.1 and 315.0 miles, respectively, above Mobile. These three locks are being built by contract, and are about half completed. They will probably be finished during 1902. The locks and abutments are being built on pile foundations, and will be of Portland cement concrete, 1 part cement, as packed in barrels, 3 parts sand, and 6 parts clean pebbles. The sand and pebbles are dredged from the river bed near the lock sites. The dams will be timber cribs with timber aprons, all founded on piles cut off about 1 ft. below low water.

Considerable dredging will be required to secure proper slack-water channel depths in the upper portions of all the pools of the Tombigbee and Warrior Systems.

Work Accomplished—Black Warrior System.

The change of name of the river, from Black Warrior to Warrior, occurs at the Tuscaloosa wagon bridge, 361.5 miles above Mobile. Locks Nos. 1, 2 and 3 are 361.9, 362.3 and 363.1 miles, respectively, above Mobile, overcoming the Tuscaloosa Falls, a series of rock rapids having a total fall of 24.5 ft. in about 2 miles. The Tuscaloosa wagon bridge is built on the lowest reef of these rapids, and here extensive channel work, largely rock excavation, partly in the Warrior and partly in the Black Warrior, was carried out in order to secure the required

depth below pool level in the lower approach to Lock No. 1, which is built on the next reef above. Considerable rock excavation was also required in the lower approach to Lock No. 3.

All three locks and dams were built with hired labor. They were begun in 1888, and completed and opened for traffic in 1895.

Lock No. 4 is 370.1 miles above Mobile, and is being built by contract. It is about half completed, and should be finished during 1902, Locks Nos. 5 to 11, inclusive, between Lock No. 4 and the junction of Mulberry and Locust Forks, have not yet been located, and no appropriation has been made for their construction.

Engineering Features—Warrior System.

Location.—Trial borings were made at every practicable site for each lock, within limits of several miles, as fixed by the profile of the river, and, at the site finally selected, borings were taken 50 ft. apart under all structures and through the approaches. Borings were made by the water-jet method, 1-in. water pipe and 2-in. casing being used. The borings usually penetrated not less than 30 ft. below the lock floor. For Locks Nos. 1, 2 and 3, 322 borings were made, aggregating 13 716 ft. in depth, at a cost of $3 861, or about 28 cents per linear foot. The results of the borings were shown on drawings and exhibited to bidders (see Fig. 3). The site of Lock No. 5, looking up stream, is shown on Plate XI, Fig. 1; Lock No. 6, with the forms for the concrete walls, is shown on Plate XI, Fig. 2.

At first, efforts were directed toward finding suitable rock reefs on which to build, but it was soon discovered from the borings that such reefs did not exist, the rock found being in fragments or in thin ledges, and generally quite soft and unsafe for foundations. Efforts were then made to find locations where pile foundations could be driven with the minimum difficulty, rock being avoided. Limits for the location of each lock were fixed so that the lower miter-sill with a given elevation would be from 2½ to 5 ft. below mean low water. Locations were sought in wide, shallow places with good, high banks, and in curved instead of straight reaches. The locks are always located on the convex shore, in order to secure better protection from drift during floods, and on straight approaches parallel to the axis of the lock re-entering the stream.

Fixed or Movable Dams.—After very careful consideration of the

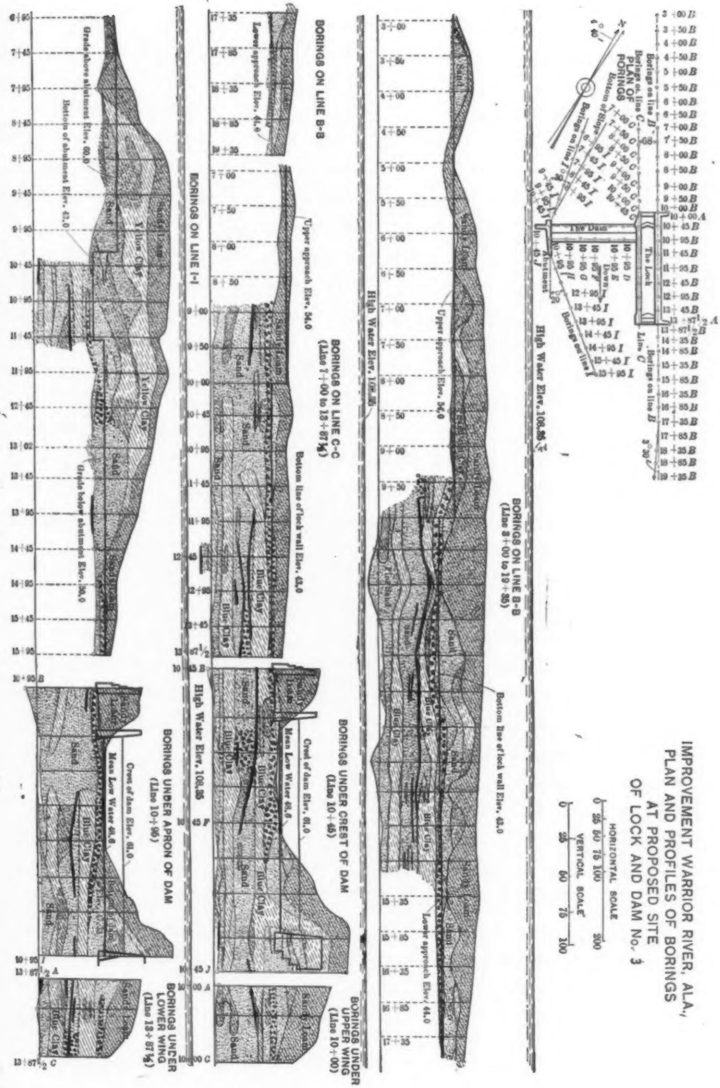

IMPROVEMENT WARRIOR RIVER, ALA.,
PLAN AND PROFILES OF BORINGS
AT PROPOSED SITE
OF LOCK AND DAM No. 3
HORIZONTAL SCALE
VERTICAL SCALE
PLAN OF BORINGS
The Dam
The Lock
Abutment
High Water Elev. 108.35
BORINGS ON LINE B-B
(Line 3+00 to 19+35)
Bottom line of lock wall Elev. 43.0
Upper approach Elev. 34.0
Lower approach Elev. 41.0
BORINGS ON LINE C-C
(Line 7+00 to 13+87½)
BORINGS UNDER CREST OF DAM
(Line 10+45)
Crest of dam Elev. 61.0
Mean Low Water 65.8
BORINGS UNDER UPPER WING
(Line 10+00)
BORINGS ON LINE B-B
BORINGS ON LINE I-I
BORINGS UNDER APRON OF DAM
(Line 10+95)
BORINGS UNDER LOWER WING
(Line 13+87½)
Sand
Sandy Loam
Blue Clay
Yellow Clay

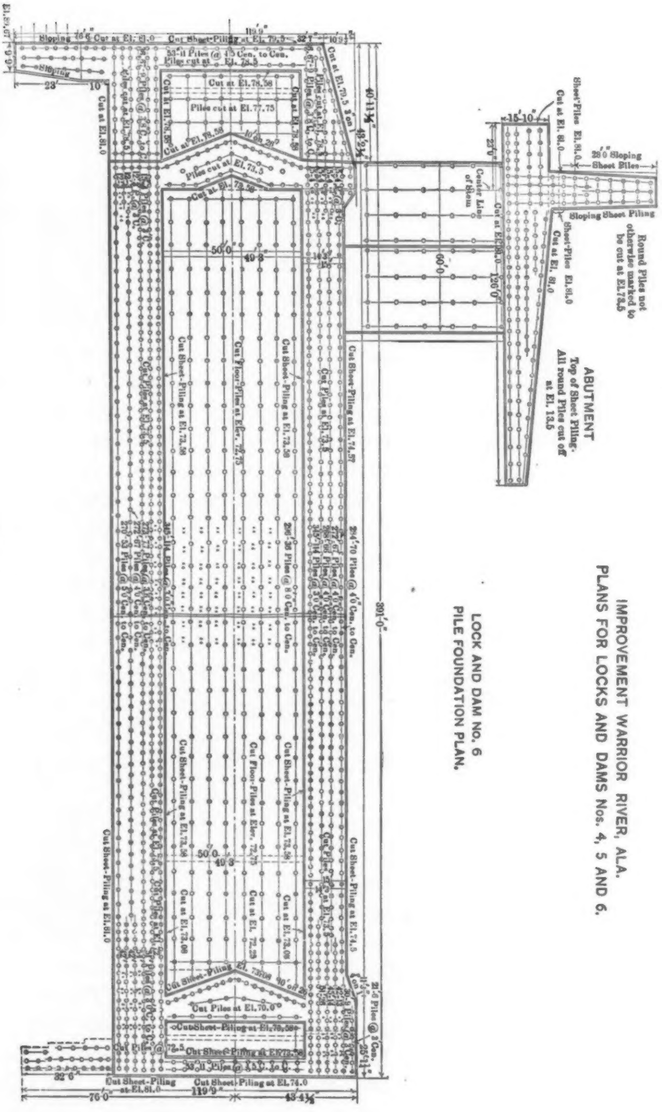
FIG. 8.

question of fixed or movable dams, the former were adopted, for several reasons. Owing to the long low-water period of about 8 months annually, and the frequent occurrence, during the other 4 months, of stages which would drown the fixed dams, movable dams would add only about 3 months annually to the period of open-river navigation. Owing to the small size of the stream, tows will always be small, and probably will not require more than one or two lockages in passing a lock. Therefore, delays from lockages will be less than on large streams where large tows are difficult to maneuver and require more lockages to pass a lock. The low-water flow of the river, at times for several months, is only 150 to 300 cu. ft. per second, and, therefore, the dams must be as tight as practicable in order to maintain the pool level at the crest of the dam. Most types of movable dams are not tight enough to keep the pools full under such conditions. The cost of construction, operation and maintenance is much greater with movable than with fixed dams; and, owing to the enormous quantities of heavy drift carried by floods, the cost of operating and maintaining movable dams on this river would probably be greater than usual. The stream when improved will serve more as a canal than as a river, and is treated accordingly.*

Lift, Guard, Etc.—The lift of each lock was fixed at 10 ft., the guard at 13 ft., and the minimum length of the dam at 300 ft. These dimensions are based largely on experience under similar conditions at Lock No. 1 of the Black Warrior System; and it is believed that they will afford practically uninterrupted navigation, the dams being drowned on a rising river by the time the lock walls are submerged. Higher lifts were not adopted because it was feared they might raise the flood heights, and thereby damage lands and other property in the valley.

Mounds.—The average width of the river at low water is about 200 ft.; therefore the length of dam adopted throws the locks and abutments well back into the banks, and causes heavy excavation, both for these structures and for the lock approaches at each site. It was not considered advisable to waste this material in the river, and therefore what is not needed for back-filling is used for building large mounds,

* Usually, one of the chief advantages of movable dams is that they reduce the height and duration of floods. In this case floods overflow the entire valley to an average depth of 5 to 10 ft. for several miles in width. It is probable, therefore, that fixed dams of 10 ft. height and 300 ft. length in the low-water channel will have little, if any, influence on the height and duration of floods.



IMPROVEMENT WARRIOR RIVER, ALA.
PLANS FOR LOCKS AND DAMS Nos. 4, 5 AND 6.

LOCK AND DAM NO. 6
PILE FOUNDATION PLAN.

FIG. 4.

5 ft. above high water, in rear of the locks and abutments. These mounds will be quite useful as places for storing property, etc., as the entire reservations, except at one lock site, are subject to overflow. The lock houses will be built on the mound back of the lock.

Plans.—Detailed drawings of Lock and Dam No. 6 are shown in Figs. 4 to 17.

Concrete.—The concrete is mixed in a 4-ft. cubical mixer making eighteen revolutions in two minutes, and the usual charge is 1 bbl. of packed cement, 3 bbls. of sand, and 6 bbls. of pebbles, which make about 29 cu. ft. of concrete rammed in place. In all, 17 688 cu. yds. of concrete and mortar facing, containing 16 973 bbls. of cement, were placed during 1901, giving an average of 1.042 cu. yds. per barrel. The sand and pebbles are quite clean, the former being coarse and sharp, and the latter about ¼ in. to 1 in. in diameter, and both showing only 33 to 34% of voids. Mortar facing, 1¼ to 1½ ins. thick, composed of 1 part packed cement and 3 parts sand, is used on all exposed faces of the walls. The walls are built in blocks about 20 ft. long and of the full thickness of the walls. The blocks are separated by continuous vertical joints from bottom to top to provide for contraction, and the joints are indicated by V-shaped grooves in the faces of the walls. Alternate blocks are built first between bulkheads, two blocks being built usually at the same time. The intermediate blocks are built after the bulkheads are removed. The concrete is well rammed, in layers about 6 ins. thick after ramming, and is usually wet enough to quake moderately when well rammed. Horizontal joints are left where the work stops at night, and are covered with ½ in. of mortar before fresh concrete is deposited next day. After the forms are taken down, rough places in the exterior finish are chiseled smooth, and, to improve the finish of the walls, several coats of a thin wash of 2 parts cement and 1 part sand are applied.

Stability of the Walls.—The lock walls are 30 ft. in height above the floor, and are built on footing courses 2 ft. wider than the bases of the walls. The faces next to the lock chamber are vertical. The bank wall is designed as a retaining wall, and its average thickness, not including the footing course, is about 37% of its height. The back is stepped so that a part of the back-filling adds to the stability of the wall against overturning. In designing the river wall it was assumed that 17 ft. is the maximum head that will come against it, this being

IMPROVEMENT WARRIOR RIVER, ALA.
PLANS FOR LOCKS AND DAMS Nos. 4, 5 AND 6.

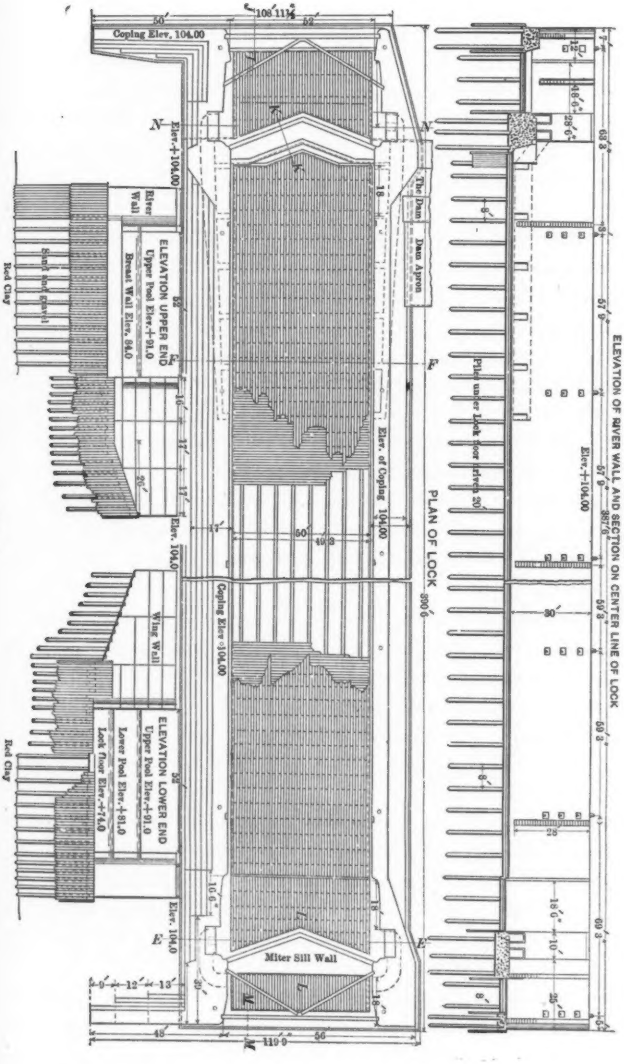


FIG. 5.

with the lower pool 10 ft. above low water and the lock chamber pumped out. The intention is never to place the coffer timbers above this height, so that should the water rise higher it will pass over the coffer timbers and relieve the head by flooding the lock chamber. Under this head the river wall has a factor of safety of 6 against overturning. With a head of 24 ft. against the river wall the factor of safety is 1.8, and with a head of 30 ft. the river wall would overturn. None of the walls can slide, owing to the nature of the pile foundations.

The buttresses are located so that the line of thrust of the gates will pass diagonally through them, but they are made thicker than necessary to take up this thrust, in order to give room for the gate-maneuvering gearing on top.

The abutment stem is a core-wall running back into the bank, with embankments on both sides. These embankments are held in place by the abutment wings, which have a face batter of 2 ins. per foot, and are designed as surcharged retaining walls. Their average thickness is about 40% of their height. The abutment also has a footing course 2 ft. wider than the bases of the walls. This is not included in computing the average thickness of the walls, and is supported by the dam and a minimum depth of $6\frac{1}{2}$ ft. of water above the footing course in the lower pool.

Pile Foundations.—Both foundation piles and sheet-piles were forced into the sand and gravel under the sites with a combination of hammer and water-jet, practically to refusal, care being taken as the penetration decreased to lessen the blow so as not to injure the pile. While the hammer was of great assistance, the jet proved to be the main dependence in getting the piles down, and they could not have been driven without it.

Every wall is entirely surrounded by an inner and outer row of Wakefield sheet-piling. The object of this is both to confine the material under the walls so that it will assist the foundation piles in sustaining their load, and to reduce the percolation of water under the walls due to difference in head. The outer row of sheet-piling also serves as part of a coffer-dam during construction.

The round foundation piles are driven 3 to 4 ft. between centers in the space between the two walls of sheet-piling. They are cut off 6 to 12 ins. above the bottom of the concrete, which is rammed around

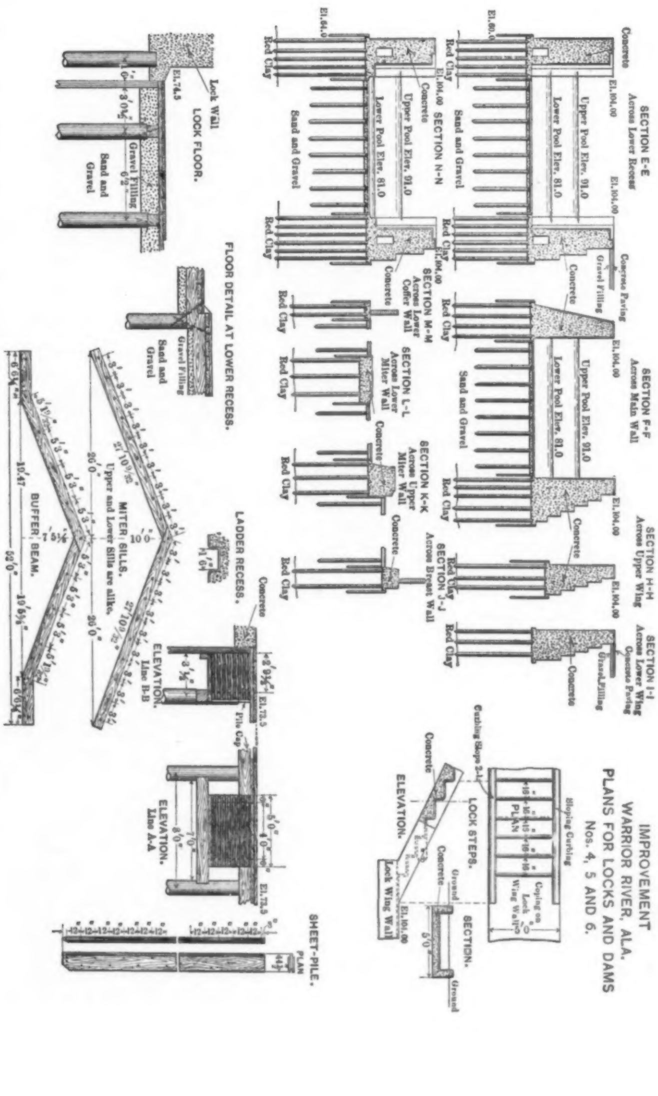

IMPROVEMENT
WARRIOR RIVER, ALA.
PLANS FOR LOCKS AND DAMS
Nos. 4, 5 AND 6.
SECTION E-E
SECTION F-F
SECTION H-H
SECTION I-I
SECTION N-N
SECTION M-M
SECTION L-L
SECTION K-K
SECTION J-J
LOCK FLOOR.
FLOOR DETAIL AT LOWER RECESS.
LADDER RECESS.
MITER SILLS.
BUFFER BEAM.
LOCK STEPS.
SHEET-PILE.

FIG. 6.

and over them. They are so close together that grillage was considered entirely unnecessary and probably detrimental to the work. The concrete, therefore, rests directly on the pile heads, from which the bark is removed. The maximum loading on the foundation piles is about 20 tons each.

The average penetration of the sheet-piles is about 12 ft., and of the round piles about 18 ft., below the bottom of the masonry. So far, not the slightest evidence of settlement has been detected. See Fig. 4.

Lock Floors.—As the floor might at times be subjected to an up-thrust, due to 17 ft. head, a tight concrete floor, safe against this up-thrust, would have to be very massive and costly. It was decided, therefore, to use a timber floor, tight against downward leakage, to avoid loss during lockages, and open for upward leakage, to avoid up-thrust. Floor-piles are driven in parallel rows and capped longitudinally with 10 x 10-in. timbers. The foundation is then covered with 15 ins. of pebbles flush with the tops of the caps and intended to permit the water to circulate freely under the floor. The caps and pebbles are then covered with a cross-layer of 3-in. plank caulked with oakum, and this with a longitudinal layer of 2-in. plank, to keep the oakum in place and protect it from culvert scour. This floor is designed for a maximum downward pressure due to 10 ft. head without aid from the pebble filling between the caps. The maximum head occurs at low water with the lock chamber filled.

The floor is relieved from up-thrust by 2-in. auger holes spaced about 10 ft. apart. This design is based on the supposition that when the lock is pumped out for repairs water under the maximum head of 17 ft. will percolate slowly under the sheet-piling and through the mass of sand and gravel under the lock. On reaching the pebble filling it will circulate more freely, pass up through the auger holes, and flow to the drainage pumps. It is believed that water due to percolation will thus be kept in motion, and cannot exert dangerous up-thrust. Downward leakage through the auger holes when the lock chamber is filled is prevented by ball-valves. See Fig. 5. The details of the valve are shown in Fig. 8.

Drain Tile.—Single lines of 8-in. drain tile, on a grade of 6 ins. per 100 ft., and emptying into the lower pool below the pool level, are laid behind the lock bank wall and the lower wing of the abut-

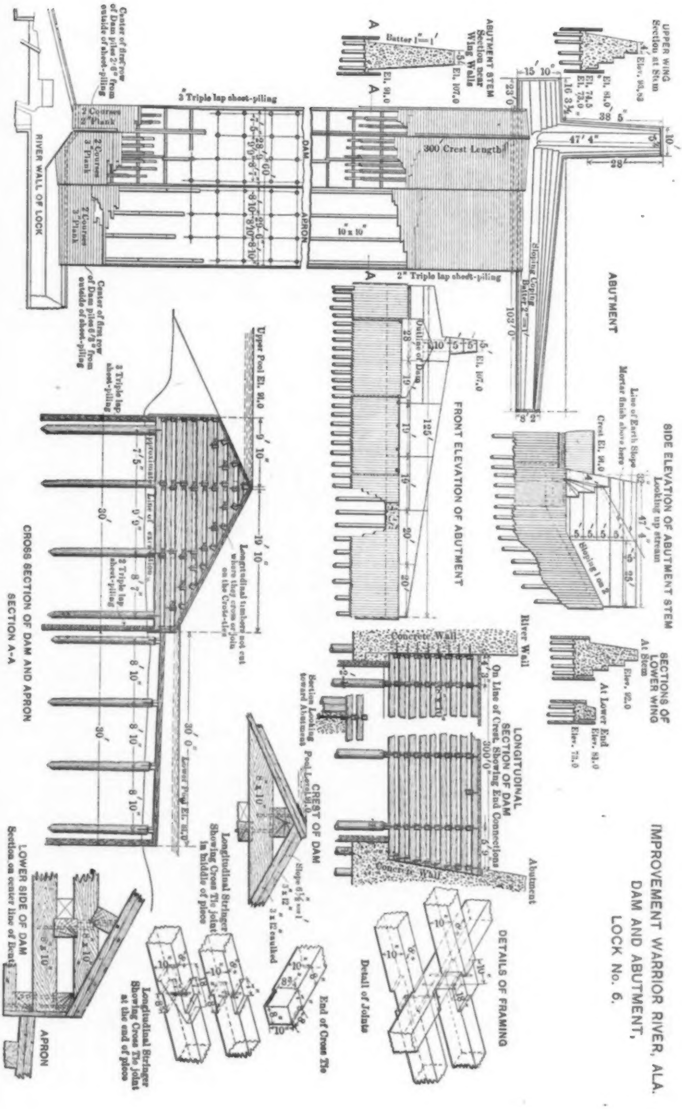

IMPROVEMENT WARRIOR RIVER, ALA.
DAM AND ABUTMENT,
LOCK No. 6.
UPPER WING
Section at Stem
ABUTMENT STEM
Section near Wing Walls
ABUTMENT
SIDE ELEVATION OF ABUTMENT STEM
Looking up stream
SECTIONS OF LOWER WING
FRONT ELEVATION OF ABUTMENT
LONGITUDINAL SECTION OF DAM
On Line of Crest, Showing End Connections
DETAILS OF FRAMING
Detail of Joints
CREST OF DAM
RIVER WALL OF LOCK
CROSS SECTION OF DAM AND APRON
SECTION A-A
LOWER SIDE OF DAM
Section on center line of Bent
APRON

FIG. 7.

ment to drain the back-filling. Vitrified tiling is used, laid with open joints and surrounded with pebbles to serve as a screen. See Fig. 10.

Dams.—Concrete dams on pile foundations were first contemplated; but this plan was abandoned because it involved a necessity for coffer-dams, difficulty in disposing of the flow of the river during construction, and the possible danger of wash-outs underneath, which would be very difficult to repair.

The plan adopted is a bottomless timber crib, resting on piles and filled with "one man" stones, the interstices in which are filled with sand and gravel washed in to reduce voids and add to the weight, but no cement is used. The crib is sheathed with plank, which is caulked to reduce leakage, and the filling rests directly on the bed of the river. The base of the dam is 30 ft. wide, and has a row of Wakefield sheet-piling under each face, to reduce percolation under the dam and prevent the escape of the filling. The crest of the dam slopes both ways to an apex 10 ft. from the up-stream face. Below the dam and under the pool level is a timber apron, 6 ins. thick and 30 ft. wide, resting on piles, and terminating with a row of Wakefield sheet-piling under the down-stream edge, to hold the filling under the apron and guard against undermining below. Should scour take place below this apron, it is proposed to dump in big stones until erosion ceases.

It is believed that this dam can be built during low water without coffer-damming, one course being placed and filled at a time, the river running over in a sheet, and the work being protected locally with flash-boards. It is also believed that, should a wash-out start under the dam, the filling will drop into and choke the cavity while small. Then the sheathing can be removed and the filling replaced in the top of the dam, which can be examined at low water by sounding through augur holes, which can be bored and afterward plugged. See Fig. 7.

Gates.—The gates are built of soft steel, of 54 000 to 64 000 lbs. ultimate strength per square inch, and designed for a fiber stress of 10 000 lbs. per square inch. They are of the mitering horizontal-girder type, with single skin, and largely follow the designs of the Deep Waterways Commission. The girders have straight down-stream flanges, in order to simplify construction, and curved up-

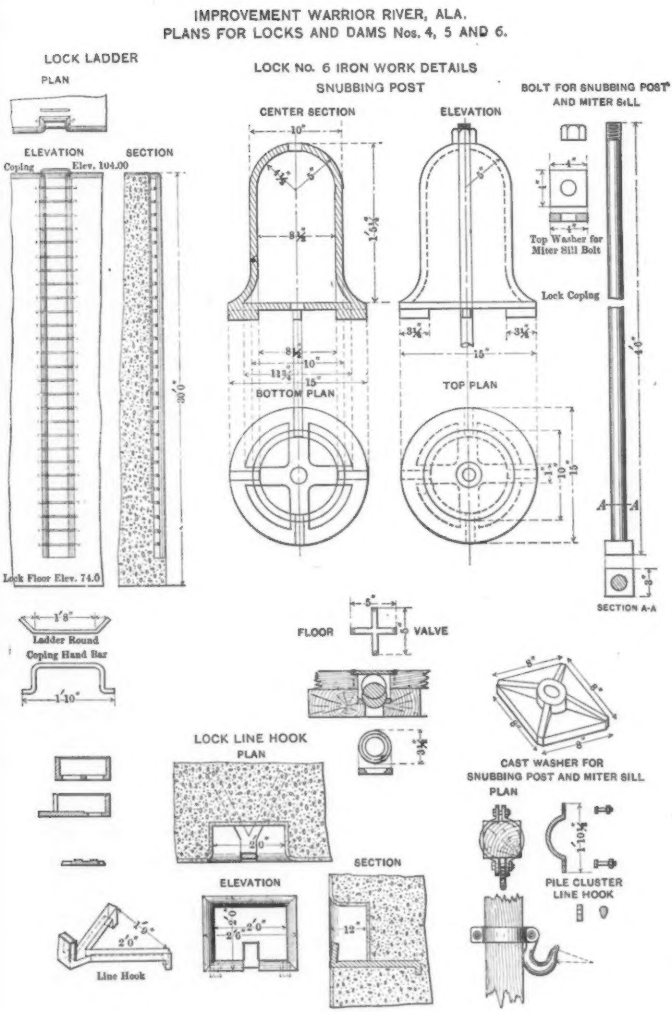

IMPROVEMENT WARRIOR RIVER, ALA.
PLANS FOR LOCKS AND DAMS Nos. 4, 5 AND 6.
LOCK LADDER
PLAN
ELEVATION
SECTION
Coping
Elev. 104.00
Lock Floor Elev. 74.0
30'0"
1'8"
Ladder Round
Coping Hand Bar
1'10"
LOCK No. 6 IRON WORK DETAILS
SNUBBING POST
CENTER SECTION
ELEVATION
10"
8¼"
1'5½"
BOTTOM PLAN
TOP PLAN
15"
BOLT FOR SNUBBING POST AND MITER SILL
Top Washer for Miter Sill Bolt
Lock Coping
SECTION A-A
FLOOR
VALVE
CAST WASHER FOR SNUBBING POST AND MITER SILL
LOCK LINE HOOK
PLAN
2'0"
ELEVATION
SECTION
12"
Line Hook
2'0"
PLAN
PILE CLUSTER LINE HOOK
1'10½"

FIG. 8.

stream flanges, in order to distribute the metal to advantage. The bottom girder bears against the miter-sill cushion, which is also straight on the bearing surface. The skin plates are $\frac{3}{8}$ in. thick, this being the minimum thickness of metal used in the gates owing to possible weakening from rust. The girders are fastened to the miter and heel posts with connecting angles. A wooden footway is bolted to the top of the gates.

The miter posts are of oak, backed with built-up steel channels to stiffen them. Wood is used for the miter-post cushions because they can be trimmed after the gates are erected, so as to adjust properly the length of the gates, and at the same time secure a perfect fit between the two posts. Also, these cushions can be easily renewed. The heel posts, owing to the difficulty of renewing wooden ones, are made of cast iron, in sections about 10 ft. long, joined with internal flanges having male and female bolted connections. Vertical dovetailed grooves are cast along the bearing surfaces of the post, and are used to hold in place strips of lead packing, which are trimmed and adjusted to make a water-tight joint with the hollow quoin. In order to avoid the wear of this packing, the heel post is hung eccentrically $\frac{1}{2}$ in., and plays off from the quoin when the gate begins to open. The quoins are cast iron, and planed true on the rubbing and bearing surfaces. The bottom of the heel post is fitted with a cast-iron socket which rests on a cast-iron pivot bolted to the masonry footing course. The bearing surfaces of the socket and pivot are chilled. The top gudgeon is a 4-in. steel pin supported by two steel plates, between which the gate yoke works. The yoke is attached to the anchorages with a gib and key which permit adjustment and also permit the removal of the yoke with ease. The yoke, etc., fits in a covered cast-iron box below the coping, where it is out of the way and yet is easily accessible for adjustment. The anchorages, which slope downward and are provided with anchor plates at the lower ends, are built into place with the concrete walls.

A stiff, built-up, operating lever is bolted to the top of the gate, and projects backward, swinging just above the coping. A small trolley travels in a slot at the outer end of the lever, and to this is attached an endless sprocket chain which passes around two idlers and a driving wheel. The latter is operated by a hand-lever and crab, the lock hand walking around the crab and pushing the lever in one

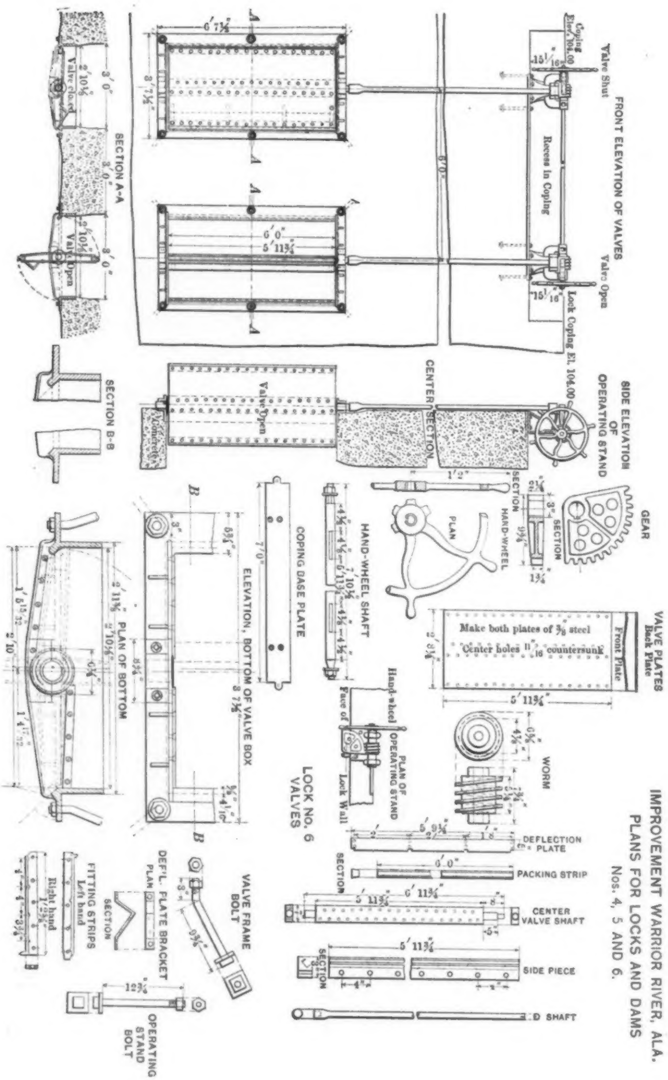

IMPROVEMENT WARRIOR RIVER, ALA.
PLANS FOR LOCKS AND DAMS
Nos. 4, 5 AND 6.
LOCK NO. 6
VALVES
FRONT ELEVATION OF VALVES
SIDE ELEVATION OF OPERATING STAND
SECTION A-A
SECTION B-B
PLAN OF BOTTOM
ELEVATION, BOTTOM OF VALVE BOX
COPING BASE PLATE
HAND-WHEEL SHAFT
PLAN OF OPERATING STAND
GEAR
HAND-WHEEL
VALVE PLATES
WORM
DEFLECTION PLATE
PACKING STRIP
CENTER VALVE SHAFT
SIDE PIECE
D SHAFT
VALVE FRAME BOLT
DEFL. PLATE BRACKET
FITTING STRIPS
OPERATING STAND BOLT
Make both plates of 3/8" steel
Center holes 11/16" countersunk

Fig. 9.

direction to open the gate and reversing the motion to close it. This hand-lever is unshipped during floods, and the remainder of the gearing, being quite close to the coping and made as flat as practicable, offers little obstruction to floating drift. This maneuvering gear is quite simple and effective. It does not easily get out of order, is quite durable, and probably compares favorably with any other method of operating lock gates by hand. See Figs. 11 and 13.

Valves.—The valves are balanced gates with vertical axes, and are built of soft steel of 54 000 to 64 000 lbs. ultimate strength. There is a steel center shaft, 3 x 7 ins., with $2\frac{3}{4}$-in. journals and two side bars of

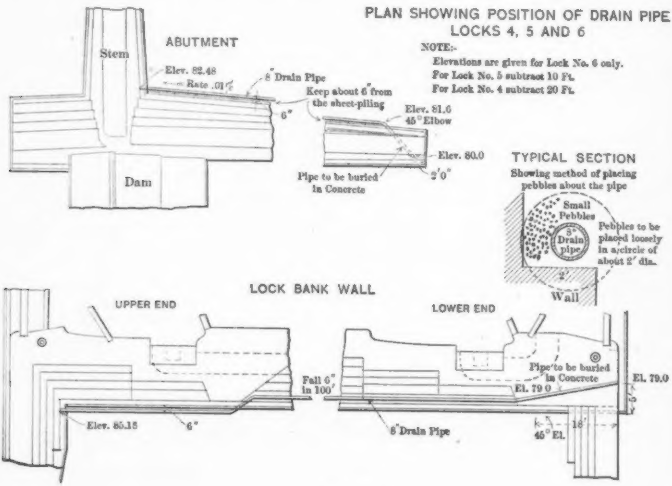


FIG. 10.

cast iron holding rubber packing strips in dovetailed grooves. The center shaft has a bottom adjustable step-bearing to reduce friction. The center shaft and side pieces are covered with two steel plates, $\frac{3}{8}$ in. thick, riveted through. The flat shaft is used to reduce the thickness of the valves, and the rivet heads are countersunk, the object being to have the open valve obstruct the flow into the culvert as little as practicable. The valve frame is a casting in one piece, built into the masonry and surrounding the valve. A steel rod, with a socket on its bottom end fitting over the square top of the valve shaft,

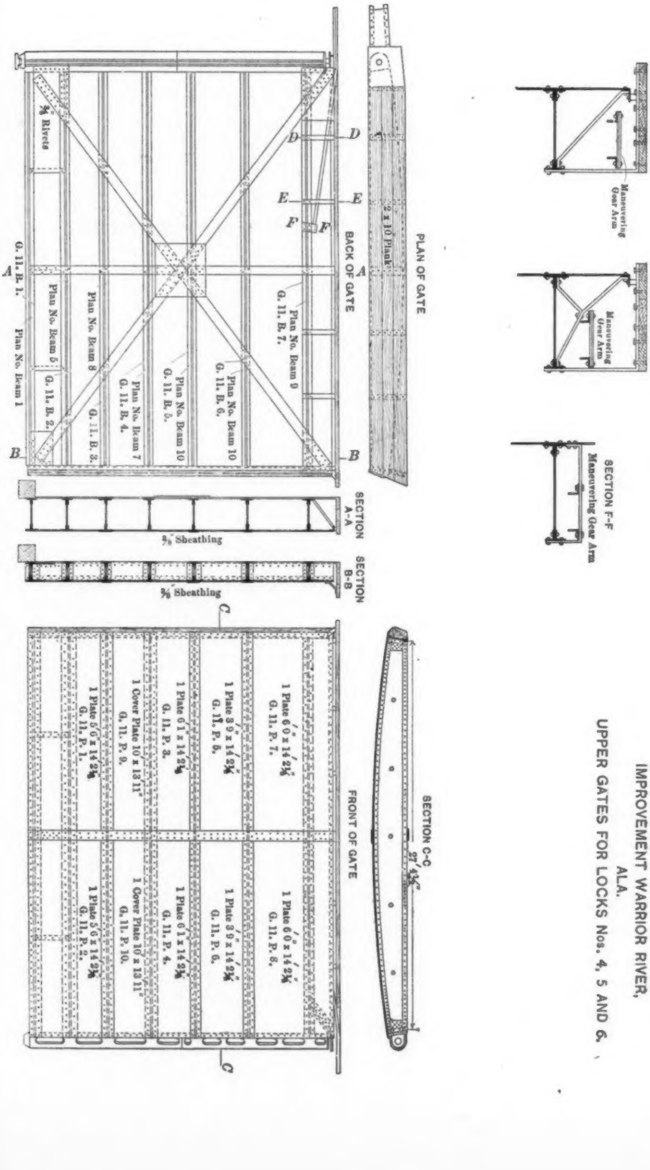

IMPROVEMENT WARRIOR RIVER,
ALA.
UPPER GATES FOR LOCKS Nos. 4, 5 AND 6.
PLAN OF GATE
BACK OF GATE
FRONT OF GATE
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
Maneuvering Gear Arm
2 x 10 Plank
⅜ Rivets
⅜ Sheathing

FIG. 11.

extends up the face of the wall to the maneuvering gear, which is placed in a recess 15 ins. below the coping in order to protect it from drift. To the top of this rod is keyed a sector worm-gear which meshes into a worm on a horizontal shaft parallel to the axis of the lock. The valves are operated in pairs from the worm shaft, which has, between the two worms, a flanged coupling which admits of adjustment so as to close the two valves exactly together. The worm shaft is operated by hand-wheels at either end, which are removed during floods, on account of drift. The remainder of the operating machinery is below or about level with the coping, and, therefore, is protected from drift.

The worm gear is used for operating the valves because it gives ample power under all conditions, and, while it acts quickly enough, it does not permit of very rapid movement of the valves, and thereby protects them from water hammer when it is necessary to close them against a head of water. Steel valves are also preferred to cast valves, because they stand shock from water hammer with much more safety than cast valves.

A steel deflection plate is fastened to the inside of the valve on the half that swings outward into the lock chamber. As soon as the valve begins to open, the reaction of the water against this plate helps to swing the valve. The plate is located so that it does not reduce the area of the waterway when the valve is fully opened. Owing to the proximity of the valves to the gates, it has been found important to have the up-stream half of the valve swing outward into the lock chamber. This gives the water a more direct flow into the culvert while the valve is swinging than if the down-stream half swung outward, and, therefore, the valve swings more easily and smoothly. The main objects sought in designing this valve were simplicity and durability, and it has proved to be quite satisfactory in practice (see Fig. 9).

Culverts.—The culverts pass through the lock walls, around the gates at each end of the lock. This avoids valves in the gates, which would tend to weaken the gates and complicate their construction. It also results in much simpler and cheaper construction than putting the culverts under the lock floor. The upper culverts extend about one-third the length of the lock chamber, and have from each culvert six different outlets into the lock chamber, at intervals of 18 ft., from center to center, in order to distribute the flow and avoid objection-

IMPROVEMENT WARRIOR RIVER, ALA. LOWER GATES FOR LOCKS Nos. 4, 5 AND 6.

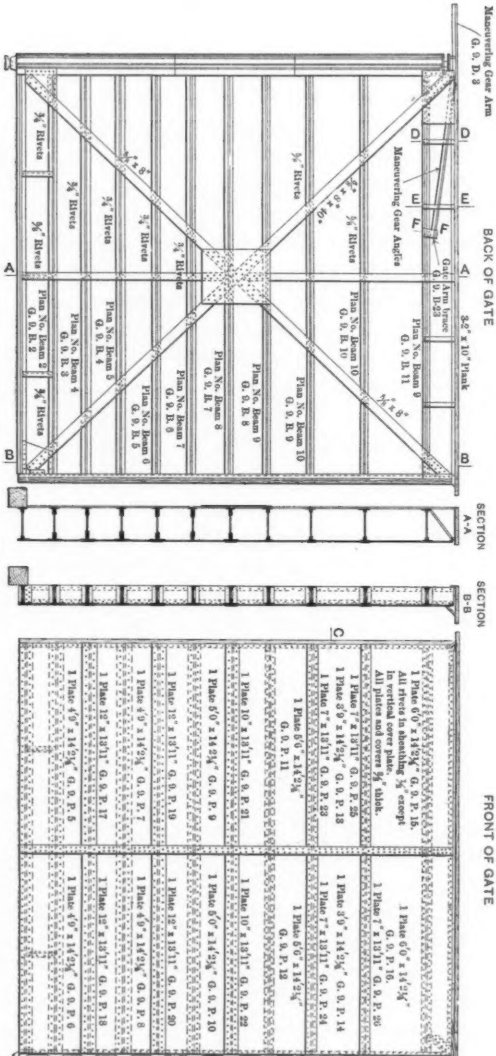


FIG. 12.

able agitation in the lock chamber. The lower culverts each have a single outlet directed so as to wash away flood deposits in the lower bay.

There are four valves at each end, giving, combined, a clear opening of 58.17 sq. ft. for filling or emptying. The two culverts at each end have a combined area of cross-section of 60 sq. ft. These dimensions, with 10 ft. lift, give an estimated time of 6.3 minutes, using c equal 0.6, for filling or emptying the lock chamber (see Figs. 14 and 15).

Coffer Timbers.—These are to be used when it is necessary to pump out for repairs, there being no guard gates provided. The timbers will be each 12 x 18 ins. x 56 ft., and will be lowered with a derrick-boat into the coffer grooves near the extreme ends of the walls, being thus built up into walls of timber 10 ft. above the lower pool and 6 ft. above the upper pool level. Triangular center posts, each having two steel channels placed so as to receive the ends of the brace timbers, will then be placed upright against the middle of the coffer timbers, and 12 x 6-in. braces will then be lowered into the center-post channels at one end and into the brace grooves at the other end. The lock chamber will then be ready for pumping out; sawdust, or cinders, or canvas being used for reducing leakage between the coffer timbers. Should the pools rise above the coffer timbers, the lock chamber will be flooded and the pressure against the river wall relieved (see Figs. 14 and 15).

Bank Protection.—The horizontal space between the bank-wall wings and the slope immediately back of these wings will be paved with concrete blocks, 6 ins. thick and about 3 ft. square, built in place on a bed of sand and gravel 6 ins. thick. The approach slopes to the lock and abutment, for the full length of the lower approaches and for 100 to 150 ft. above the dam, will be protected with hand-placed rip-rap, 12 ins. thick, of hard, durable stone. It is expected that this rip-rap will slip in spots while floods are receding, and will have to be repaired from time to time until it becomes covered with a thick growth of willows and other vegetation.

Pile Clusters.—Heart piles will be driven in clusters above and below the locks to guide vessels in entering. Each cluster will contain eight piles, seven cut off level with the coping of the lock, and a center pile projecting 18 ins. above and rounded for a snubbing post. The center piles in certain clusters will also be equipped with line

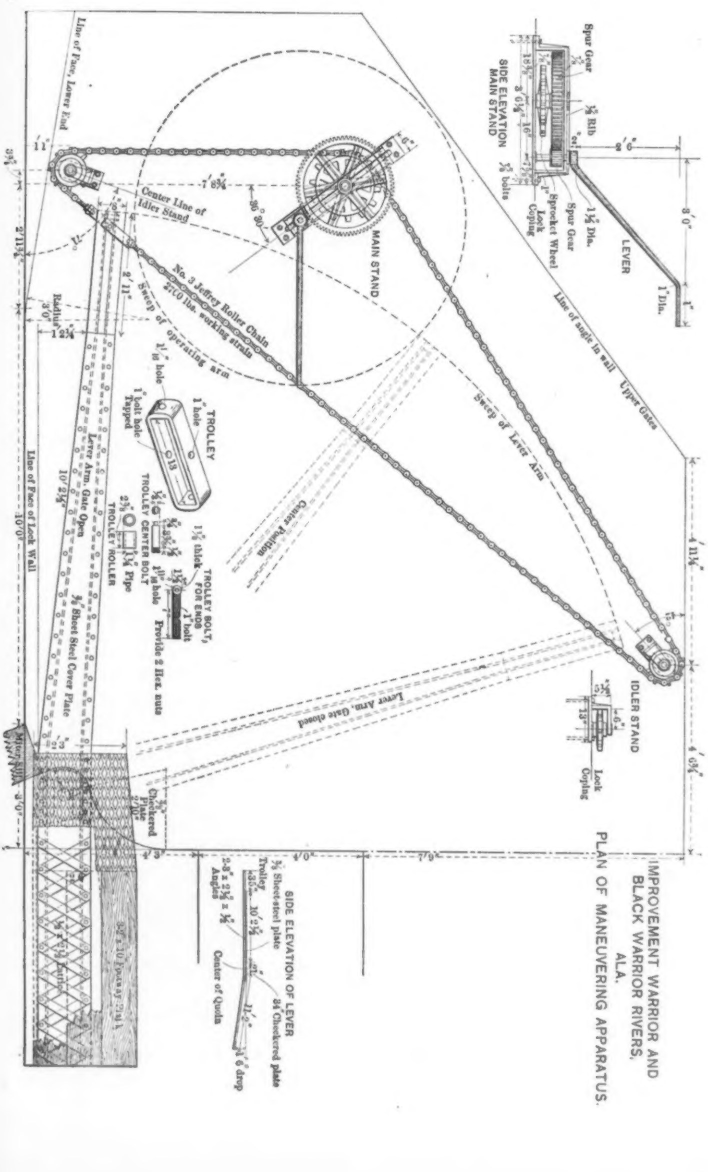

IMPROVEMENT WARRIOR AND
BLACK WARRIOR RIVERS,
ALA.
PLAN OF MANEUVERING APPARATUS.
SIDE ELEVATION
MAIN STAND
Spur Gear
Sprocket Wheel
Lock Coping
LEVER
MAIN STAND
No. 3 Jeffrey Roller Chain
2700 lbs. working strain
Center Line of Idler Stand
Line of Face, Lower End
Sweep of operating arm
Sweep of Lever Arm
Line of angle in wall
Upper Gates
Center Position
TROLLEY
TROLLEY CENTER BOLT
TROLLEY ROLLER
TROLLEY BOLT, FOR ENDS
Provide 2 Hex. nuts
Lever Arm, Gate Open
Lever Arm, Gate closed
Line of Face of Lock Wall
IDLER STAND
Checkered Plate
SIDE ELEVATION OF LEVER
Center of Quoin
Miter Sill

FIG. 13.

hooks for use at varying stages of water. That part of each pile above low water will be all heart, squared, 12 x 12 ins., with rounded corners. Below low water the piles will be left round. There will be six clusters above the lock, the first four being in line with the river wall and spaced 30 ft. between centers. The other two clusters flare from this line 10°, and are spaced 90 ft. between centers. The greater spacing of these two clusters is to facilitate the passage of drift. The other clusters are spaced closer, because the suck of the dam is greater near the lock. There will be four clusters below the lock in line with the bank wall, the first cluster 30 ft. from the end of lock wall and the others spaced 60 ft. from center to center. These clusters will keep vessels in proper position for entering the lock, and barges can be tied to them when fleets are locked in sections (see Fig. 16).

Engineering Features—Black Warrior System.

Plans.—Sections and plans of Dams Nos. 1, 2 and 3, and detailed drawings of Lock and Dam No. 4, are shown in Figs. 17 to 20. The site of Lock No. 4, looking down stream, is shown on Plate XII, Fig. 1; the lower end of Lock No. 1, and Dam No. 1, are shown on Plate XII, Fig. 2.

Lift, Guard, Etc.—The lift at Lock No. 1 will be 10 ft. when the next dam below is built. The guard is 13 ft. and the length of the dam 339 ft. The lift of Lock No. 2 is 8.5 ft., the guard is 15.5 ft., and the length of the dam 409 ft. The lift at Lock No. 3 is 10.5 ft., the guard is 15 ft., and the length of the dam 650 ft. Generally, these dams are navigable when the lock walls are submerged. In 1897, during one of the most rapid rises which has occurred since the dams were built, the fall over the dams, just as the lock walls became submerged, was observed as follows: Dam No. 1, 1.14 ft. fall, river rising 2 ft. per hour; Dam No. 2, 2.10 ft. fall, river rising 2 ft. per hour; and Dam No. 3, 1.40 ft. fall, river rising 1 ft. per hour. In each case the dams became navigable in a few hours after the locks were submerged. While this same rise was subsiding, just as the lock walls emerged, the fall over Dams Nos. 1, 2 and 3 was 0.10 ft., 0.15 ft., and 0.20 ft., respectively.

The lift at Lock No. 4 will be 12 ft., the guard 12 ft., and the length of the dam 640 ft. Owing to the character and steepness of the stream here and above, it was not considered practicable to make the dams

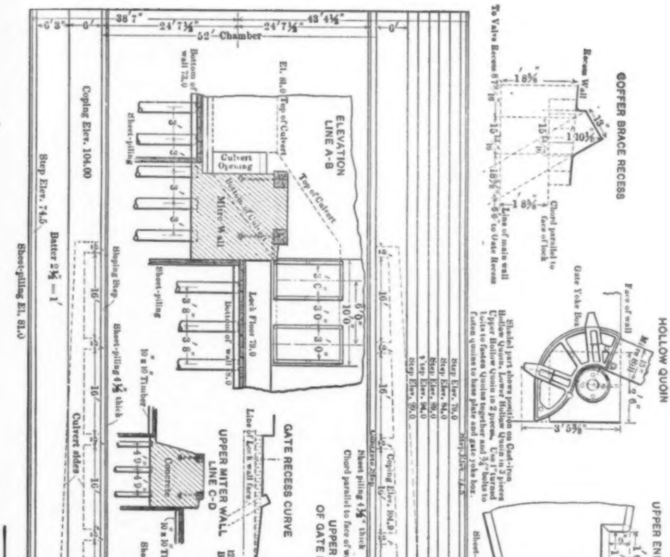

IMPROVEMENT WARRIOR RIVER,
LOCKS 4, 5 AND 6.
UPPER END OF LOCK.
COFFER BRACE RECESS
HOLLOW QUOIN
UPPER END OF RIVER WALL
UPPER END OF BANK WALL
ELEVATION LINE A-B
GATE RECESS CURVE
UPPER MITER WALL LINE C-D
UPPER END OF GATE RECESS
BREAST WALL & COFFER TIMBERS
Coping Elev. 104.00
Elev. Miter Sill 83.0
Breast Wall Elev. 84.0
Step Elev. 79.5
52′ Chamber

FIG. 14.

navigable when the locks are submerged. The lock walls will be about 10 ft. below ordinary high water and about 19 ft. below extreme flood, and will never be submerged except for a few days at a time. During such times the river would be unsafe for navigation, under any circumstances, and all traffic should seek harbors of refuge.

Masonry.—The walls and dams are built of sandstone quarried in the vicinity of the locks along the banks of the river and in the river bed. The stone for Lock and Dam No. 3 was quarried from the river bed just below the lock under what is now the channel for vessels. This quarry was in a reef just above falls about 7 ft. high, and was exposed by building a dam and training wall out from the shore and down to the falls. It covered about 2 acres of the river bed, and was 12 to 18 ft. deep. It was operated during the low-water period, only, and was drained with two 3-in. pulsometers. It yielded a hard, durable sandstone at a very reasonable cost, as there was no stripping.

All masonry is founded on bed-rock, the exposed faces of the lock and abutment walls being built of cut stone and the interior and back, where not exposed, of uncut rubble. The foundations for the faces of the lock walls and culverts are excavated to the grade of the lock floor, the rear of the walls being stepped up on the original bed-rock, only loose and soft rock being removed. Weep holes for drainage are left in the lower wing of the abutment and in the lower wing of the lock bank wall; also, a portion of the rear of the lock bank wall is built of dry rubble thoroughly bonded into the mortar part of the wall. This serves as a drain for the back-filling, and reduces the cost of the wall without materially reducing its weight. Pointing is done as the work progresses, so that the pointing and other mortar may set together.

A small quantity of Louisville natural cement, 1 of cement to 2 of sand, was used in beginning Lock No. 1. With this exception, Portland cement has been used throughout. At Locks Nos. 1, 2 and 3 the cement was measured loose and the face stones were set in mortar composed of 1 of cement and 3 of sand. Some of the rubble was built up in similar mortar, some was grouted, and some had the interstices filled with concrete composed of 1 of cement, 3 of sand and 5 of broken stone, rammed in place. At Lock No. 4 the cement is measured packed, and the face stones are set in mortar composed of 1 of cement and 3 of

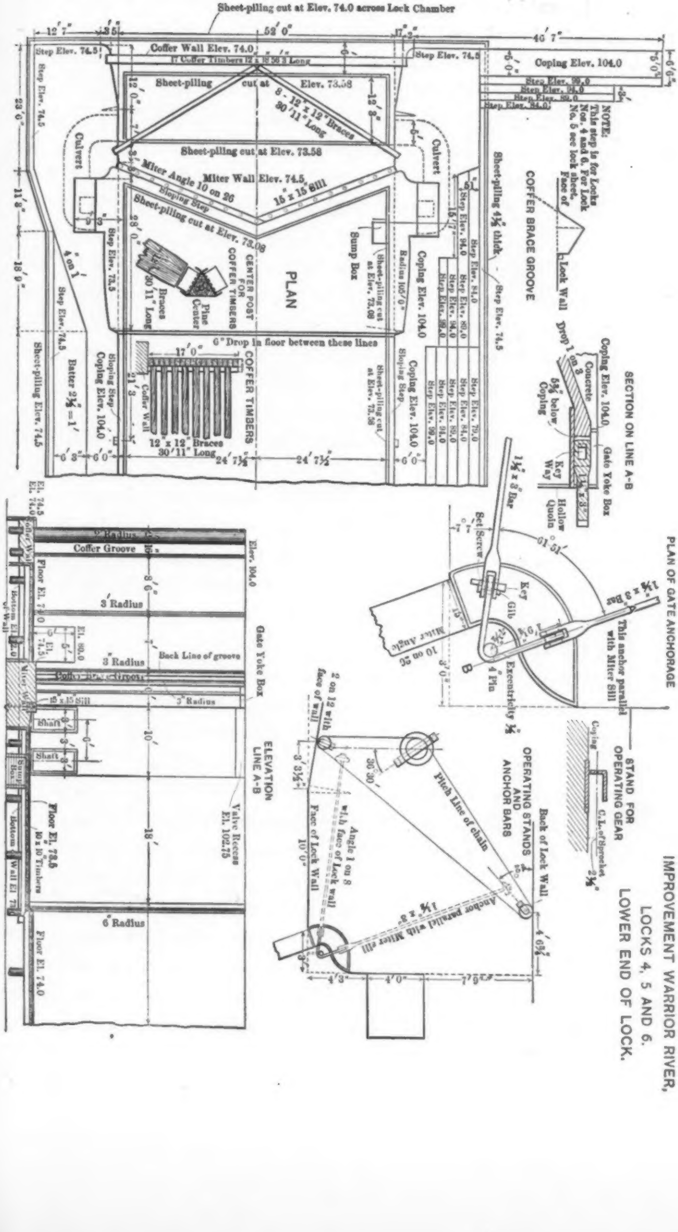


FIG. 15.

IMPROVEMENT WARRIOR RIVER, LOCKS 4, 5 AND 6. LOWER END OF LOCK.

sand. The rubble is bedded and built up in concrete composed of 1 of cement, 3 of sand, and 5 of clean pebbles, the concrete being thoroughly rammed into the interstices. The stability of the walls is about the same as those for the Warrior locks. See Fig. 20.

Dams.—Movable dams were not considered for the Black Warrior System, because open-river navigation is not practicable on this portion of the stream under any conditions. Dams Nos. 1, 2 and 3 are of the rock-fill type, without mortar or core-wall. The down-stream face is composed of large roughly dressed stones, laid in steps and dowelled together. A timber crib is built into the up-stream face and sheathed. A timber wale is anchor-bolted to the top course of dressed stones, and a sloping crest of timber is drift-bolted to this wale and to the top of the crib in the up-stream face. These three dams were built during the low-water seasons, without coffer-damming. A floating derrick was used above the dams, and stationary derricks erected on the bed-rock reefs were used below the dams and were moved ahead as the building progressed. The stone for Dams Nos. 1 and 2 was delivered by barge, and for Dam No. 3 by rail, a track being laid on stone-filled cribs along the toe of the dam. The low-water flow of the stream ran through the rock-fill dams until all the stone-work and the timber crest were in place. Then the vertical sheathing on the up-stream face and the filling above the dams were placed, and the pools filled in a short time.

These dams are very cheap, but not altogether satisfactory. The filling above the dams is not permanent, and has to be renewed from time to time. The leakage is so great that at times it is difficult to keep the pools full. On certain stages there is a strong "back lash" below the dams, which holds and tosses drift sometimes for a day or two before it can get away. Saw logs, and other heavy drift, at such times, if they come back "end on" in the "back lash," dive when they reach the sheet of water falling over the crest, and strike the step stones in the down-stream face of dams like so many battering rams. This action of the drift made several large breaches in the down-stream faces of the dams, there being no dowels in the face stones when the dams were first built. Afterward, two 1¼-in. dowel pins were put through each face stone into the stones below. No large breaches, but several small ones, have occurred since the face stones were dowelled. The crests of these dams have settled as much

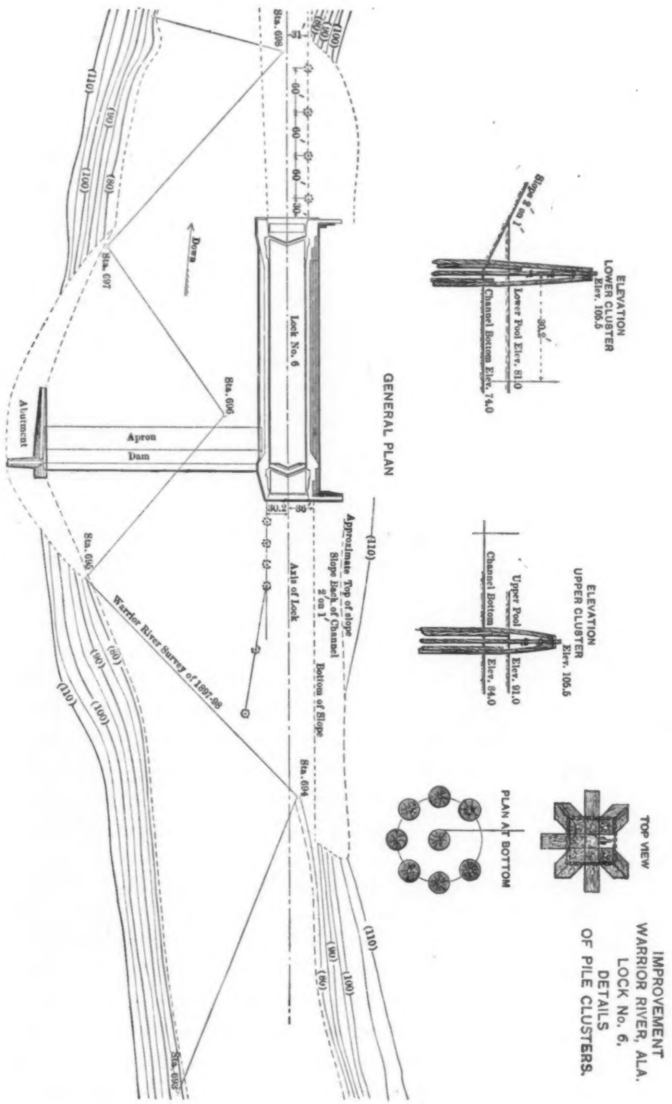

IMPROVEMENT
WARRIOR RIVER, ALA.
LOCK No. 6.
DETAILS
OF PILE CLUSTERS.
GENERAL PLAN
ELEVATION
LOWER CLUSTER
Elev. 105.5
Lower Pool Elev. 81.0
Channel Bottom Elev. 74.0
Slope 2 on 1
ELEVATION
UPPER CLUSTER
Elev. 105.5
Upper Pool Elev. 91.0
Channel Bottom Elev. 84.0
TOP VIEW
PLAN AT BOTTOM
Lock No. 6
Apron
Dam
Abutment
Down
Axis of Lock
Approximate Top of slope
Slope Back of Channel 2 on 1
Bottom of Slope
Warrior River Survey of 1897-98
Sta. 698
Sta. 697
Sta. 696
Sta. 695
Sta. 694
Sta. 693

FIG. 16.

as 4 ins. in places, and the bed-rock, below the dams which have no aprons, is wearing from the action of the water and drift. It is probably just a question of time when these dams will have to be rebuilt or extensively repaired. See Fig. 17.

The objections to this type of dam led to a different design for Dam No. 4, which is being built of rubble masonry in mortar and concrete, but with timber coping to reduce the cost. The face stones are laid in mortar composed of 1 of packed cement and 3 of sand. The interior stones are bedded and built up in concrete composed of 1 of packed cement, 3 of sand and 5 of clean pebbles. Both faces are carefully pointed as the work progresses. The down-stream face is built of large stones roughly dressed, with a straight batter of 6 ins. per foot. It is thought that this type of dam will be tight and will suffer little injury from drift playing below; also, the down-stream toe is carried well down into bed-rock, which will have to wear to the bottom of the toe before the dam can begin to undermine. Thus the bed-rock will be utilized as an apron, and should it wear enough to endanger the dam it can be replaced with an artificial apron. It is probable that a concrete dam of ogee section would be better and cheaper for such conditions, and will be built with the locks projected above Dam No. 4. See Fig. 18.

Gates.—The gates for Locks Nos. 1, 2 and 3 are built of steel, and are of the mitering, horizontal-girder type, with a single skin of ¼-in. plates on the water side. The girders are 20-in. I-beams, with a ¾-in. camber against the water pressure. The beams are all the same size, the spacing decreasing from the top to the bottom of the gate to provide for the increase of pressure, and are fastened to the miter and heel posts with connecting angles. The bottom beam bears against the miter-sill cushion, the bearing surface of which has a ¾-in. camber to fit the beam.

The miter-posts are of oak, backed with a steel plate ⅝ in. thick. The heel posts are of cast iron, and are similar to those for the Warrior locks, but larger in diameter, owing to the greater depth of the ends of the girders. The top gudgeon is of soft steel, and is turned and pressed into the top of the heel post, which is flush with the coping. The yoke, therefore, comes above the coping, and is connected with turnbuckles to the anchorages, which are bolted to the top of the wall and partly countersunk. The operating lever is

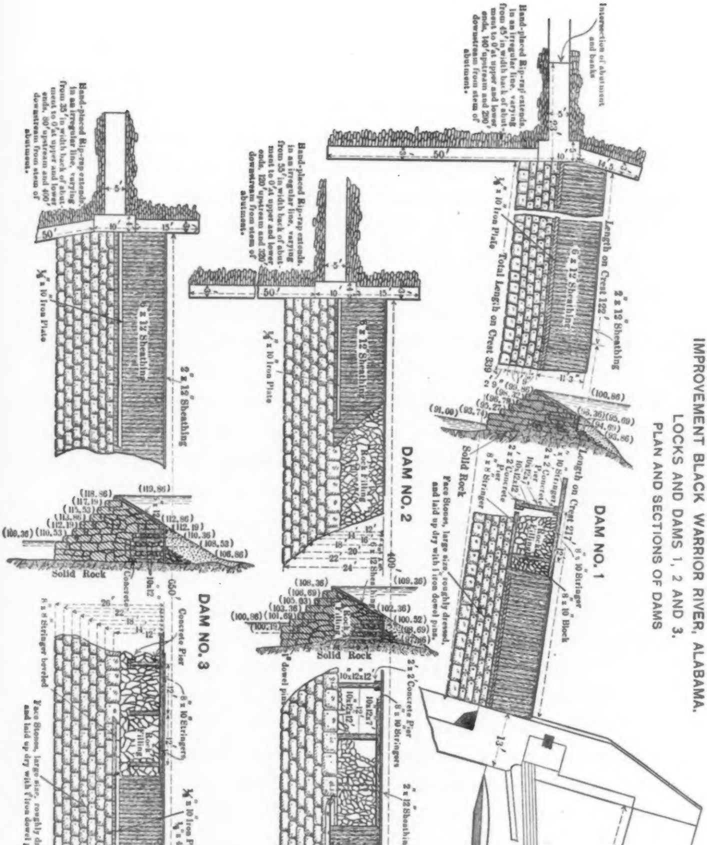

IMPROVEMENT BLACK WARRIOR RIVER, ALABAMA.
LOCKS AND DAMS 1, 2 AND 3.
PLAN AND SECTIONS OF DAMS
DAM NO. 1
DAM NO. 2
DAM NO. 3
Solid Rock
Center Line of Lock

FIG. 17.

fastened to the gate above the yoke, and slopes downward so that the trolley slot at the outer end hangs close to the coping. Other features of these gates are similar to those of the Warrior locks.

The gates for Lock No. 4 are similar in design to those for the Warrior locks, except that the anchorages are countersunk and bolted to the top of the wall, instead of being built into the masonry.

Valves.—Two kinds of valves were tried at Lock No. 1. One was a flexible curtain valve of asbestos rubber-cloth, stiffened on the culvert side with horizontal iron slats slightly separated and riveted to the cloth. This valve was lifted from the bottom, peeling up, instead of rolling, by means of gearing on the walls and counterweights working in wells. These valves were used for several years, but did not prove durable or satisfactory, and were finally replaced with valves similar to those for the Warrior locks.

The other valve used at Lock No. 1 is a cast-iron, balanced gate with a vertical axis. The casting is in one piece, with horizontal stiffening ribs, and is cast around the steel center shaft, the ends of which are turned for journals. The frame consists of oak sills bolted into recesses in the masonry and lined with wrought-iron valve bearings. The valve revolves in cast journal boxes bolted to the oak sills. A steel rod, with a socket on the bottom end fitting over the square top of the valve shaft, extends up to the top of the wall, where it is revolved to open or close the valve by a simple lever fastened to the top of the rod. This same valve is in use at Locks Nos. 2 and 3, and has proved fairly satisfactory. The journal boxes and frame linings work loose, and wear more or less, and several of the valves were cracked shortly after being erected, probably due to water hammer caused by closing the valves suddenly against a head of water. Also, the valve cannot be operated as smoothly with the single lever as with the worm gear. The steel valve operated by the worm gear is considered, on the whole, safer and more durable.

The valves for Lock No. 4 are similar in design to those for the Warrior locks.

Coffer Timbers, Etc.—When Locks Nos. 1, 2 and 3 were built, light needle-dams, extending about 2 ft. above the pool level, were provided at the upper ends, but no provision for coffer-dams was made at the lower ends. The erection of the needle-dam exposes the upper gates and valves, which are entirely above the lower pool level. When repairs

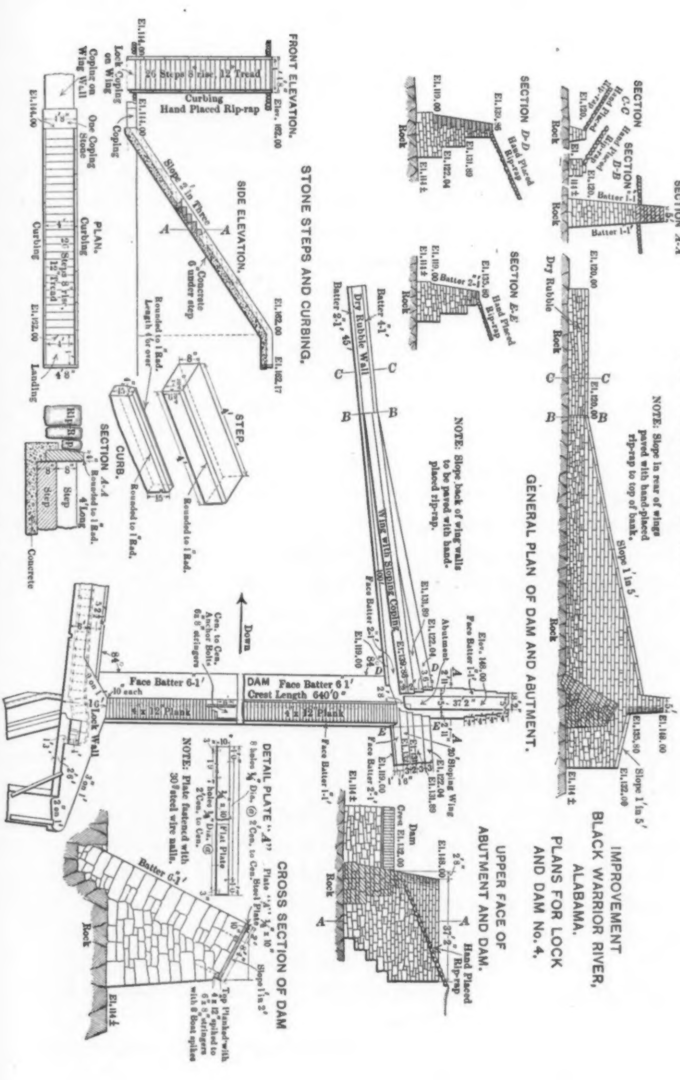

IMPROVEMENT
BLACK WARRIOR RIVER,
ALABAMA.
PLANS FOR LOCK
AND DAM No. 4.
FRONT ELEVATION OF ABUTMENT.
NOTE: Slope in rear of wings paved with hand-placed rip-rap to top of bank.
GENERAL PLAN OF DAM AND ABUTMENT.
NOTE: Slope back of wing walls to be paved with hand-placed rip-rap.
UPPER FACE OF
ABUTMENT AND DAM.
CROSS SECTION OF DAM
DETAIL PLATE "A"
NOTE: Plate fastened with 30d steel wire nails.
DAM Face Batter 6-1'
Crest Length 640'0"
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
STONE STEPS AND CURBING.
FRONT ELEVATION.
SIDE ELEVATION.
PLAN.
STEP.
CURB.
SECTION A-A

FIG. 18.

are needed to the lower gates and valves, or when the lock chambers need cleaning out, there being no dredge available, the practice has been to draw each pool down at low water through the lock below, raise the needle-dam at the upper end, put in a low, cheap, temporary coffer-dam of clay or canvas at the lower end, and pump out. The objection to this method is that it is only available at low water, and a very slight rise will flood the pool below, while a rise of over 2 ft. will flood the needle-dam above. This renders repair work quite precarious. Also, in order to drain a pool, it is necessary to lash open the gates of the lock below and turn the river through the lock. This leaves the lock in bad condition in case of a sudden rise. One gate has recently been greatly injured under these circumstances by being torn from its lashings and slammed against the miter-sill by the current. It had to be taken down, repaired, and re-erected.

The pools below Dams Nos. 2 and 3 are quite short, and can be drained quickly at low water. So far, there has been an open river below Dam No. 1, but the next dam below, Warrior No. 6, will probably be built during 1902. This will make a pool 46 miles long below Dam No. 1, which cannot be drained quickly, or without damage to commerce, even at low water. Therefore, during 1901, provision was made at the lower end of Lock No. 1 for a permanent coffer-dam, 5 ft. above the pool level and otherwise similar to those for the Warrior locks. Similar coffer-dams will probably be provided, later, at the lower ends of Locks Nos. 2 and 3.

The needle-dams in use at Locks Nos. 1, 2 and 3 are erected on masonry breast walls extending across the heads of the locks and up to within about 7 ft. of the pool level. The needles are each 4 ins. wide, and are placed against two sills. The bottom sill is built and anchored into the masonry of the breast wall. The top sill rests at each end in recesses cut in the faces of the lock walls, and is supported at intervals of about 7½ ft. by steel trestles anchored to the breast wall and hinged so that when not in use they lie flat upon it. The needles, when in place, slope down stream at an angle of about 20° from the vertical. These needle-dams are quite simple and efficient, and are easily maneuvered. The principal objection to them is that they extend only 2 ft. above the pool level.

At Lock No. 4, coffer timbers at both ends, similar to those for the Warrior locks, are provided.

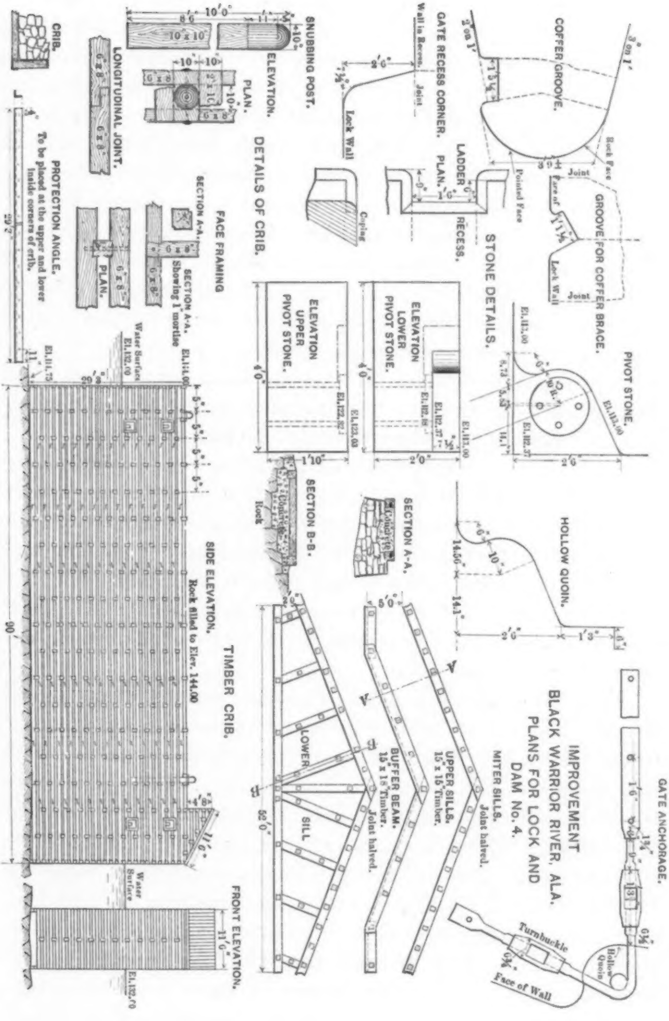

IMPROVEMENT
BLACK WARRIOR RIVER, ALA.
PLANS FOR LOCK AND
DAM No. 4.
STONE DETAILS.
COFFER GROOVE.
GROOVE FOR COFFER BRACE.
PIVOT STONE.
HOLLOW QUOIN.
GATE ANCHORAGE.
Turnbuckle
Face of Wall
GATE RECESS CORNER.
LADDER RECESS.
ELEVATION LOWER PIVOT STONE.
ELEVATION UPPER PIVOT STONE.
SECTION A-A.
SECTION B-B.
MITER SILLS.
UPPER SILLS.
BUFFER BEAM.
LOWER SILL
DETAILS OF CRIB.
SNUBBING POST.
ELEVATION.
PLAN.
LONGITUDINAL JOINT.
FACE FRAMING
CRIB.
PROTECTION ANGLE.
To be placed at the upper and lower inside corners of crib.
TIMBER CRIB.
SIDE ELEVATION.
Rock filled to Elev. 144.00
FRONT ELEVATION.

FIG. 19.

Bank Protection.—At Locks Nos. 1, 2 and 3 all bank protection consists of hand-placed rip-rap, 12 ins. thick. It covers the horizontal space between the bank-wall wings and most of the slope back of these wings, and the sloping banks for a short distance above and below each lock; also the sloping banks for short distances above and below each lock ; also the sloping banks for a short distance above and for about 300 ft. below each abutment. Immediately back of the locks and abutments this protection extends well up toward the top of the slope, but tapers down toward the low-water line at the lower end. This rip-rap slips in places, after floods, and requires more or less repairs each season, but, on the whole, has served its purpose well. In places it is now covered with willows and other vegetation, which help to hold it in position. The banks here consist largely of clay, and do not erode as readily as the alluvial banks of the Warrior.

The bank protection at Lock No. 4 will be similar to that at Locks Nos. 1, 2 and 3, except that well-shaped stone paving, 15 ins. thick, will be used on the horizontal space between the bank-wall wings.

Guard Cribs.—When Locks Nos. 1, 2 and 3 were built, a single guard crib, 90 ft. long and 11½ ft. wide, was placed above and in line with the river wall at each lock, a space of 10 ft. being left between the crib and the lock wall for the passage of drift. The top of the crib was level with the coping of the lock, and a sloping drift guard on the upper end of the crib extended 5 ft. higher. The cribs were built of 6 x 8-in. yellow pine, with cross-pieces at intervals of 5 ft., drift-bolted together, and filled with "one man" stone.

At certain stages the suck of the dam causes a strong current around the head of the crib. This caused the wreck of a towboat and several barges, in February, 1901, they being drawn over the dam while maneuvering to enter the head of Lock No. 2. To afford better protection to traffic, four additional cribs, 24 ft. long and 12 ft. wide, with 21-ft. openings between them for drift, are now being built above each of the old cribs. The new cribs are on a line flaring 10° at Locks Nos. 1 and 2, and 15° at Lock No. 3, toward the river from the line of the old crib, and extend the protection to a total distance of 280 ft. above the head of lock. It is probable that, should fleets ever be handled that will require two or more lockages to pass a lock, guard cribs with snubbing posts will be required below and in line with the bank walls, but none have yet been built or provided for.

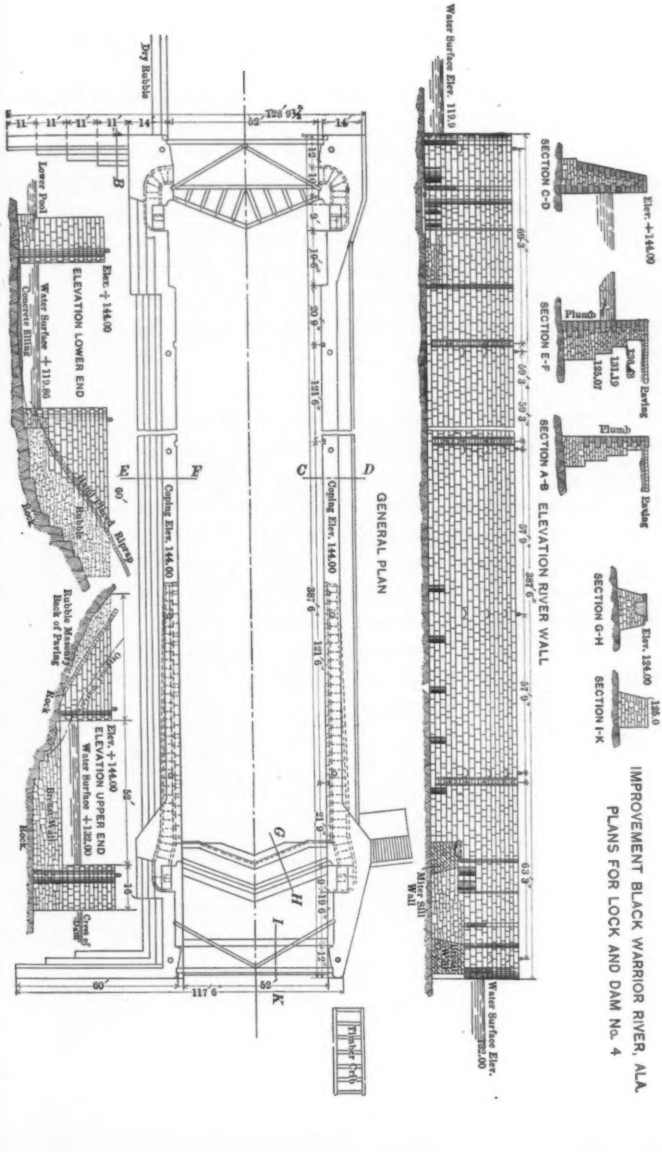

IMPROVEMENT BLACK WARRIOR RIVER, ALA.
PLANS FOR LOCK AND DAM No. 4
GENERAL PLAN
ELEVATION RIVER WALL
SECTION C-D
SECTION E-F
SECTION A-B
SECTION G-H
SECTION I-K
ELEVATION LOWER END
ELEVATION UPPER END
Coping Elev. 144.00
Water Surface Elev. 119.9
Dry Rubble
Lower Pool
Timber Crib
Miter Sill Wall
Hand Placed Riprap
Rubble Masonry Back of Paving
Concrete Filling
Rock
Crest of Dam

FIG. 20.

A single crib, similar to those first built at Locks Nos. 1, 2 and 3, is provided for at Lock No. 4, but it is probable that additional cribs will be needed there also. (See Fig. 19.)

Other Features.—With given lifts, the location of each lock is largely fixed by the profile of the stream, and there is little choice between different sites. As far as practicable, the locks are placed on a convex shore, to secure protection from drift, and straight approaches. No mounds have been built at the Black Warrior locks, because there has been no surplus excavation and the river banks are much higher than at the Warrior locks. The lock floors consist of bed-rock, as left after excavating to grade, and are quite rough. The culverts and other details not mentioned are similar to those described for the Warrior locks.

Cost.

As stated previously, the total cost of these improvements will approximate $5 000 000, or $250 000 per lock and dam.

Tombigbee System.—The work on Lock and Dam No. 1 is about three-fifths completed. No detailed estimates of the cost of completing this and building Locks and Dams Nos. 2 and 3 have yet been made. The cost of concrete in place at Lock No. 1 was about $6.34 per cubic yard, under fair working conditions, but this did not include the cost of coffer-dams and other incidental expenses.

Warrior System.—The estimated cost of Locks and Dams Nos. 1, 2 and 3 is $874 000, as itemized in Table No. 2.

TABLE No. 2.

Lock No. 1. Kind of work and material.	Measure.	Quantity.	Price.	Amount.
Concrete	Cubic yards.	12 000	$7.50	$90 000
Excavation	"	260 000	0.35	91 000
Gravel filling	"	1 000	1.00	1 000
Stone filling	"	7 000	2.50	17 500
Rip-rap	"	4 000	2.50	10 000
Puddling	"	100	1.50	150
Miter-sills	Feet, B. M.	4 000	50.00	200
Dam timber	"	320 000	30.00	9 600
Apron timber	"	110 000	25.00	2 750
Floor timber	"	116 000	40.00	4 640
Sheathing	"	20 000	20.00	400
Sheet-piling	"	550 000	40.00	22 000
Foundation piles	Linear feet.	50 000	0.30	15 000
Heart piles	"	6 000	0.35	2 100
Drain pipes	"	500	0.50	250
Grubbing and clearing	Complete.			1 000
Coffer-dams				10 000
				$277 590
Purchase of sites				2 000
Gates, valves and irons				15 000
Buildings				5 000
Engineering and incidentals				33 410
Total				$333 000
Lock No. 2.				
Concrete	Cubic yards.	12 000	$7.50	$90 000
Excavation	"	135 000	0.35	47 250
Gravel filling	"	1 000	1.00	1 000
Stone filling	"	4 000	2.50	10 000
Rip-rap	"	2 900	2.50	7 250
Puddling	"	100	1.50	150
Miter-sills	Feet, B. M.	4 000	50.00	200
Dam timber	"	195 000	30.00	5 850
Apron timber	"	66 000	25.00	1 650
Floor timber	"	116 000	40 00	4 640
Sheathing	"	20 000	20.00	400
Sheet-piling	"	465 000	40.00	18 600
Foundation piles	Linear feet.	48 000	0.30	14 400
Heart piles	"	6 000	0.35	2 100
Drain pipe	"	500	0.50	250
Grubbing and clearing	Complete.			1 000
Coffer-dams				8 000
				$212 740
Purchase of sites				500
Gates, valves and irons				15 000
Buildings				5 000
Engineering and incidentals				26 760
Total				$260 000

TABLE No. 2—(*Continued*).

LOCK No. 3. Kind of work and material.	Measure.	Quantity.	Price.	Amount.
Concrete	Cubic yards.	12 000	$7.50	$90 000
Excavation	"	175 000	0.35	61 250
Gravel filling	"	1 000	1.00	1 000
Stone filling	"	3 600	2.50	9 000
Rip-rap	"	3 600	2.50	9 000
Puddling	"	100	1.50	150
Miter sills	Feet, B. M.	4 000	50.00	200
Dam timber	"	195 000	30.00	5 850
Apron timber	"	66 000	25.00	1 650
Floor timber	"	116 000	40.00	4 640
Sheathing	"	20 000	20.00	400
Sheet-piling	"	520 000	40.00	20 800
Foundation piles	Linear feet.	55 000	0.30	16 500
Heart piles	"	6 000	0.35	2 100
Drain pipe	"	500	0.50	250
Grubbing and clearing	Complete.			1 000
Coffer-dams				8 000
				$231 790
Purchase of sites				500
Gates, valves and irons				15 000
Buildings				5 000
Engineering and incidentals				28 710
Total				$281 000
RECAPITULATION.				
Lock No. 1				$333 000
Lock No. 2				260 000
Lock No. 3				281 000
Grand total				$874 000

The bids received for Locks and Dams Nos. 4, 5 and 6 are shown in Table No. 3.

TABLE No. 3.

Lock and Dam No. 4.

Items.	Measure.	Quantity.	No. 1.		No. 2.		No. 3.		No. 4.		No. 5.	
			Price.	Amount.	Price.	Amount.	Price.	Amount.	Price.	Amount.	Price.	Amount.
Concrete	Cubic yards.	11 300	$7.50	$84 750.00	$11.00	$124 300.09	$9.50	$107 350.00	$6.48	$73 224.00	$5.95	$67 235.00
Excavation	"	140 000	0.50	70 000.00	0.40	56 000.00	0.50	70 000.00	0.35	49 000.00	0.28	39 200.00
Gravel filling	"	3 900	0.75	2 925.00	1.00	3 900.00	1.00	3 900.00	1.00	3 900.00	0.42	1 638.00
Puddling	"	100	1.50	150.00	1.25	125.00	1.25	125.00	2.00	200.00	0.60	60.00
Mattress work	Square yards.	6 200	0.90	5 580.00	1.00	6 200.00	1.00	6 200.00	0.50	3 100.00	0.48	2 976.00
Miter-sills	Feet, B. M.	4 000	44.00	176.00	50.00	200.00	45.00	180.00	50.00	200.00	36.00	144.00
Dam timber	"	155 000	29.00	4 495.00	40.00	6 200.00	32.50	5 037.00	30.00	4 650.00	29.00	4 495.00
Apron timber	"	67 000	24.50	1 641.50	27.50	1 842.50	30.00	2 010.00	25.00	1 675.00	19.20	1 286.40
Floor timber	"	112 000	30.00	3 360.00	32.50	3 640.00	35.00	3 920.00	35.00	3 920.00	40.00	4 480.00
Sheathing	"	20 000	20.00	400.00	50.00	1 000.00	30.00	600.00	20.90	400.00	17.00	340.00
Sheet-piling	"	275 000	50.00	13 750.00	60.00	16 500.00	40.00	11 000.00	50.00	13 750.00	50.00	13 750.00
Foundation piles	Linear feet.	44 000	0.32	14 080.00	0.40	17 600.00	0.25	11 000.00	0.20	8 800.00	0.29	12 760.00
Heart piles	"	2 000	0.34	680.00	1.00	2 000.00	0.50	1 000.00	0.25	500.00	0.30	600.00
Drain pipe	"	500	1.00	500.00	0.60	300.00	0.28	140.00	0 50	250.00	0.50	250.00
Grubbing and clearing	Complete.	....	....	200.00	....	5 000.00	......	4 600.00	......	1 000.00	......	240.00
Totals	....	....	....	$202 687.50	....	$244 807.50	......	$227 062.50	......	$164 569.00	......	$149 454.40
Alternative quantities:												
Stone filling	Cubic yards.	3 100	$2.00	$6 200.00	$5.00	$15 500.00	$2.50	$7 750.00	$3.75	$11 625.00	$1.14	$3 534.00
Stone paving	"	1 150	7.00	8 050.00	5.50	6 325.00	6.50	7 475.00	3.75	4 312.50	2.70	3 105.00
Rip-rap (hand-placed)	"	2 100	2.50	5 250.00	5.25	11 025.00	3.50	7 350.00	3.25	6 825.00	1.20	2 520.00

Mattress work	Square yards.	5 000	0.90	4 500.00	1.00	5 000.00	1.00	5 000.00	0.50	2 500.00	0.48	2 400.00
Miter sills	Feet, B. M.	4 000	44.00	176.00	50.00	200.00	45.00	180.00	50.00	200.00	36.00	144.00
Dam timber	"	155 000	29.00	4 495.00	35.00	5 425.00	32.50	5 037.50	30.00	4 650.00	29.00	4 495.00
Apron timber	"	67 000	24.50	1 641.50	25.00	1 675.00	30.00	2 010.00	25.00	1 675.00	19.20	1 286.40
Floor timber	"	112 000	30.00	3 360.00	30.00	3 360.00	35.00	3 920.00	35.00	3 920.00	40.00	4 480.00
Sheathing	"	20 000	20.00	400.00	50.00	1 000.00	30.00	600.00	20.00	400.00	17.00	340.00
Sheet-piling	"	275 000	50.00	13 750.00	60.00	16 500.00	40.00	11 000.00	50.00	13 750.00	47.00	12 925.00
Foundation piles	Linear feet.	55 000	0.32	17 600.00	0.50	27 500.00	0.25	13 750.00	0.20	11 000.00	0.29	15 950.00
Heart piles	"	2 100	0.34	714.00	1.00	2 100.00	0.50	1 050.00	0.25	525.00	0.30	630.00
Drain pipe	"	500	1.00	500.00	0.60	300.00	0.28	140.00	0.50	250.00	0.50	250.00
Grubbing and clearing	Complete.			200.00		4 000.00		4 600.00		1 000.00		240.00
Totals				$172 486.50		$219 985.00		$190 212.50		$147 046.00		$135 640.40
Alternative quantities:												
Stone filling	Cubic yards.	3 200	$2.00	$6 400.00	$4.00	$12 800.00	$2.50	$8 000.00	$3.50	$11 200.00	$1.14	$3 648.00
Stone paving	"	950	7.00	6 650.00	4.50	4 275.00	6.50	6 175.00	3.50	3 325.00	2.70	2 565.00
Rip-rap (hand-placed)	"	1 700	2.50	4 250.00	4.25	7 225.00	3.50	5 950.00	3.00	5 100.00	1.20	2 040.00

Summary.

Totals of bids for Lock and Dam No. 4		$202 687.50		$244 807.50		$227 062.50		$164 569.00		$149 454.40
Totals of bids for Lock and Dam No. 5		162 856.50		204 072.50		182 632.50		145 179.00		132 135.40
Totals of bids for Lock and Dam No. 6		172 486.50		219 985.00		190 212.50		147 046.00		135 640.40
Total bid for all three locks and dams		$538 030.50		$668 865.00		$599 907.50		$456 794.00		$417 230.20

The work was let to the lowest bidders, at the prices given in bid No. 5. The latest estimate of the total cost of these three locks and dams is $616 793, itemized as follows:

Contract with lowest bidders	$458 869
Purchase of sites	1 524
Gates, valves and irons	45 400
Buildings	15 000
Engineering and incidentals	96 000
Total	$616 793

This will make a total cost of $1 490 793, for the Warrior System of six locks and dams.

TABLE No. 3—(*Continued*).

Lock and Dam No. 5.

Items.	Measure.	Quantity.	No. 1. Price.	No. 1. Amount.	No. 2. Price.	No. 2. Amount.	No. 3. Price.	No. 3. Amount.	No. 4. Price.	No. 4. Amount.	No. 5. Price.	No. 5. Amount.
Concrete....................	Cubic yards.	12 300	$7.50	$92 250.00	$10.00	$123 000.00	$9.00	$110 700.00	$6.48	$79 704.00	$6.00	$73 800.00
Excavation..............	"	45 000	0.50	22 500.00	0.50	22 500.00	0.50	22 500.00	0.50	22 500.00	0.32	14 400.00
Gravel filling............	"	4 400	0.75	3 300.00	1.00	4 400.00	1.00	4 400.00	1.00	4 400.00	0.42	1 848.00
Puddling	"	100	1.50	150.00	1.25	125.00	1.25	125.00	2.00	200.00	0.60	60.00
Mattress work...........	Square yards.	3 600	0.90	3 240.00	1.00	3 600.00	1.00	3 600.00	0.50	1 800.00	0.48	1 728.00
Miter-sills	Feet, B. M.	4 000	44.00	176.00	50.00	200.00	45.00	180.00	50.00	200.00	36.00	144.00
Dam timber	"	171 000	29.00	4 959.00	37.50	6 412.50	32.50	5 557.50	30.00	5 130.00	29.00	4 959.00
Apron timber...........	"	67 000	24.50	1 641.50	25.00	1 675.00	30.00	2 010.00	25.00	1 675.00	19.20	1 286.40
Floor timber	"	112 000	30.00	3 360.00	30.00	3 360.00	35.00	3 920.00	35.00	3 920.00	40.00	4 480.00
Sheathing...............	"	20 000	20.00	400.00	50.00	1 000.00	30.00	600.00	20.00	400.00	17.00	340.00
Sheet-piling..............	"	270 000	50.00	13 500.00	50.00	13 500.00	40.00	10 800.00	50.00	13 500.00	50.00	13 500.00
Foundation piles.........	Linear feet.	50 000	0.32	16 000.00	0.40	20 000.00	0.25	12 500.00	0.20	10 000.00	0.29	14 500.00
Heart piles..............	"	2 000	0.34	680.00	1.00	2 000.00	0.50	1 000.00	0.25	500.00	0.30	600.00
Drain pipe...............	"	500	1.00	500.00	0.60	300.00	0.28	140.00	0.50	250.00	0.50	250.00
Grubbing and clearing..	Complete.	....	....	200.00	....	2 000.00	......	4 600.00	......	1 000.00	......	240.00
Totals............	....	....	....	$162 856.50	....	$204 072.50	......	$182 632.50	......	$145 179.00	......	$132 135.40
Alternative quantities:												
Stone filling...........	Cubic yards.	3 600	$2.00	$7 200.00	$5.00	$18 000.00	$2.50	$9 000.00	$3.60	$12 960.00	$1.14	$4 104.00
Stone paving....	"	1 650	7.00	11 550.00	5.50	9 075.00	6.50	10 725.00	3.60	5 940.00	2.70	4 455.00
Rip-rap (hand-placed)	"	1 200	2.50	3 000.00	5.25	6 300.00	3.50	4 200.00	3.10	3 720.00	1.20	1 440.00

Lock and Dam No. 6.

Items.	Measure.	Quantity.	No. 1. Price.	No. 1. Amount.	No. 2. Price.	No. 2. Amount.	No. 3. Price.	No. 3. Amount.	No. 4. Price.	No. 4. Amount.	No. 5. Price.	No. 5. Amount.
Concrete................	Cubic yards.	11 200	$7.50	$84 000.00	$10.00	$112 000.00	$9.00	$100 800.00	$6.48	$72 576.00	$6.00	$67 200.00
Excavation..............	"	76 000	0.50	38 000.00	0.50	38 000.00	0.50	38 000.00	0.40	30 400.00	0.31	23 560.00
Gravel filling............	"	4 000	0.75	3 000.00	0.70	2 800.00	1.00	4 000.00	1.00	4 000.00	0.42	1 680.00
Puddling.................	"	100	1.50	150.00	1.25	125.00	1.25	125.00	2.00	200.00	0.60	60.00

Black Warrior System.—The total cost of Locks and Dams Nos. 1, 2 and 3, including local office expenses, but not including office expenses at the district headquarters in Mobile, was $541 215.57, itemized as in Table No. 4.

TABLE No. 4.

Lock and Abutment No. 1.

(Lock masonry, 9 762 cu. yds.; abutment masonry, 325 cu. yds.)

Item.	Measure.	Quantity.	Cost.	Rate per unit.
Stone quarried	Cubic yards.	10 087	$34 400.96	$3.410+
Stone cut	"	3 530	37 637.27	10.662+
Masonry laid, including cement	"	10 087	25 474.07	2.525+
Earth excavation	"	10 809	3 016.99	0.279+
Rock excavation	"	3 778	5 014.38	1.327+
Earth filling	"	4 500	1 145.65	0.254+
Rock filling	"	2 500	1 238.98	0.495+
Paving, 12 ins. thick	Square yards.	6 774	2 199.52	0.222+
Turfing	"	3 132		
Gates and valves			12 010.50	
Coffer-dam and pumping			8 555.54	
Handling and hauling			20 273.22	
Boats and buildings			4 518.44	
Track and roads			10 882.29	
Tools and plant			28 535.18	
Engineering and superintendence			19 404.90	
Incidentals			6 771.01	
Total			$221 078.90	

Dam No. 1.

(Dam, 10 ft. high, 339 ft. long, and 21 ft. width of base.)

Item.	Measure.	Quantity.	Cost.	Rate per unit.
Lumber and iron	Feet, B. M.	36 238	$1 010.65	$27.889+
Carpenter work	"	36 238	336.30	9.280+
Stone quarried	Cubic yards.	1 467	5 003.09	3.410+
Stone, rough-dressed	"	360	1 059.41	2.942+
Masonry laid, dry rubble	"	1 467	1 243.11	0.847+
Earth excavation	"	329	60.04	0.182+
Filling above dam	"	502	425.00	0.846+
Handling and hauling			465.22	
Track and roads			100.13	
Tools and plant			554.90	
Engineering and superintendence			477.04	
Incidentals			143.73	
Total			$10 878.62	

Guard Crib No. 1.

(Guard crib, 29 ft. 10 ins. high, 90 ft. long, and 11 ft. 8 ins. wide.)

Item.	Measure.	Quantity.	Cost.	Rate per unit.
Lumber and iron	Feet, B. M.	34 453	$470.40	$13.653+
Carpenter work	"	34 453	239.07	6.939+
Filling rock	Cubic yards.	1 640	567.51	0.346+
Total			$1 276.98	

TABLE No. 4—(*Continued*).

Lock and Abutment No. 2.

(Lock masonry, 10 723 cu. yds.; abutment masonry, 559 cu. yds.)

Item.	Measure.	Quantity.	Cost.	Rate per unit.
Stone quarried	Cubic yards.	11 282	$16 509.14	$1.463+
Stone cut	"	3 595	25 744.53	7.161+
Masonry laid, including cement	"	11 282	21 330.53	1.890+
Earth excavation	"	6 876	1 636.01	0.237+
Rock excavation	"	941	995.57	1.057+
Earth filling	"	6 552	1 351.86	0.206+
Paving, 12 ins. thick	Square yards.	8 367	2 039.52	0.217+
Turfing	"	1 021		
Gates and valves			9 698.45	
Coffer-dam and pumping			2 402.10	
Handling and hauling			10 002.62	
Boats and buildings			3 441.87	
Track and roads			9 867.23	
Tools and plant			22 512.68	
Engineering and superintendence			13 841.16	
Incidentals			5 507.40	
Total			$146 880.67	

Dam No. 2.

(Dam, 13 ft. high, 410 ft. long, and 24 ft. width of base.)

Lumber and iron	Feet, B. M.	53 556	$850.91	$15.888+
Carpenter work	"	53 556	381.80	7.128+
Stone quarried	Cubic yards.	3 646	3 350.85	0.919+
Stone, rough-dressed	"	373	631.37	1.692+
Masonry laid, dry rubble	"	3 646	1 868.20	0.512+
Earth excavation	"	360	45.85	0.127+
Filling above dam	"	607	401.99	0.662+
Cement			135.00	
Handling and hauling			586.54	
Track and roads			106.48	
Tools and plant			964.03	
Engineering and superintendence			1 081.62	
Incidentals			497.61	
Total			$10 902.25	

Guard Crib No. 2.

(Guard crib, 28 ft. 8 ins. high, 90 ft. long, and 11 ft. 6 ins. wide.)

Lumber and iron	Feet, B. M.	33 109	$419.80	$12.679+
Carpenter work	"	33 109	226.17	6.831+
Filling rock	Cubic yards.	1 105	263.38	0.238+
Total			$909.35	

Lock and Abutment No. 3.

(Lock masonry, 10 885 cu. yds.; abutment masonry, 530 cu. yds.)

Stone quarried	Cubic yards.	11 415	$18 923.33	$1.657+
Stone cut	"	3 997	24 057.88	6.018+
Masonry laid, including cement	"	11 415	20 691.09	1.812+
Earth excavation	"	3 529	910.35	0.257+
Rock excavation	"	2 500	2 501.68	1.000+
Earth filling	"	7 496	2 156.41	0.287+
Paving, 12 ins. thick	Square yards.	7 036	966.43	0.137+
Gates and valves			9 919.30	
Handling and hauling			8 510.01	
Boats and buildings			3 445.21	
Track and roads			2 022.80	
Tools and plant			17 852.08	
Engineering and superintendence			13 892.26	
Incidentals			5 712.64	
Total			$131 561.47	

TABLE No. 4—(*Concluded*).

Dam No. 3.

(Dam, 15 ft. high, 650 ft. long, and 26 ft. width of base.)

Item.	Measure.	Quantity.	Cost.	Rate per unit.
Lumber and iron	Feet, B. M.	93 759	$1 274.55	$13.593+
Carpenter work	"	93 759	689.77	7.356+
Stone quarried	Cubic yards.	6 130	4 153.50	0.677+
Stone, rough-dressed	"	898	1 403.10	1.171+
Masonry laid, dry rubble	"	6 130	1 916.10	0.312+
Filling above dam	"	1 444	868.85	0.601+
Cement			135.00	
Handling and hauling			1 168.34	
Track and roads			635.48	
Tools and plant			2 668.77	
Engineering and superintendence			1 278.22	
Incidentals			145.15	
Total			$16 336.83	

Guard Crib No. 3.

(Guard crib, 28 ft. 8 ins. high, 90 ft. long, and 11 ft. 6 ins. wide.)

Item.	Measure.	Quantity.	Cost.	Rate per unit.
Lumber and iron	Feet, B. M.	33 109	$468.97	$14.164+
Carpenter work	"	33 109	417.95	12.623+
Filling rock	Cubic yards.	1 090	503.58	0.462+
Total			$1 390.50	

SUMMARY.

Cost of Lock and Dam No. 1	$233 234.50
Cost of Lock and Dam No. 2	158 692.27
Cost of Lock and Dam No. 3	149 288.80
Grand total	$541 215.57

The unit cost of Lock and Dam No. 3 was considerably below that of Nos. 1 and 2, due to favorable conditions and the rapid prosecution of the work. The masonry was begun on June 15th, 1893, and the lock walls were finished on November 8th, 1893. The entire masonry of the lock was completed, ready for the gates and valves, on December 26th, 1893.

Analyzing the cost of Lock and Abutment No. 3 gives the percentages shown in Table No. 5.

TABLE No. 5.

Items.	Cost.	Percentage.
Earth excavation	$910.35	0.7
Paving	966.43	0.7
Track and roads	2 022.80	1.5
Earth filling	2 156.41	1.6
Rock excavation	2 501.68	1.9
Boats and buildings	3 445.21	2.6
Incidentals	5 712.64	4.4
Handling and hauling	8 510.01	6.5
Gates and valves	9 919.30	7.4
Engineering and superintendence	13 892.26	10.6
Tools and plant	17 852.08	13.6
Quarrying	18 923.33	14.4
Cement and laying stone	20 691.09	15.8
Stone cutting	24 057.88	18.3
Totals	$131 561.47	100

The following items do not properly enter into the cost of the masonry:

Earth excavation	$910.35
Rock excavation	2 501.68
Earth filling	2 156.41
Paving	966.43
Gates and valves	9 919.30
Lock tender's house	1 705.76
	$18 159.93

This gives a total cost of $113 401.54 for 11 415 cu. yds. of all classes of masonry, or $9.934 + per cubic yard.

Analyzing the cost of Dam No. 3 gives the percentages shown in Table No. 6.

TABLE No. 6.

Items.	Cost.	Percentage.
Cement	$135.00	0.8
Incidentals	145.15	0.9
Track and roads	635.48	3.9
Carpenter work	689.77	4.2
Filling above dam	868.85	5.3
Handling and hauling	1 168.34	7.2
Lumber and iron	1 274.55	7.8
Engineering and superintendence	1 278.22	7.9
Dressing stone	1 403.10	8.6
Laying stone	1 916.10	11.7
Tools and plant	2 668.77	16.3
Quarrying	4 153.50	25.4
Totals	$16 336.83	100

TABLE No. 7.

Items.	Measure.	Quantity.	No. 1.		No. 2.		No. 3.		No. 4.		No. 5.	
			Price.	Amount.	Price.	Amount.	Price.	Amount.	Price.	Amount.	Price.	Amount.
Quoins	Cubic yards.	80	$22.00	$1 760.00	$25.00	$2 000.00	$22.00	$1 760.00	$22.00	$1 760.00	$25.00	$2 000.00
Coping	"	300	18.00	5 400.00	20.00	6 000.00	18.80	5 640.00	20.00	6 000.00	18.00	5 400.00
Pointed-face masonry	"	2 240	12.00	26 880.00	19.00	42 560.00	13.50	30 240.00	15.00	33 600.00	15.50	34 720.00
Rock-face masonry	"	1 560	10.00	15 600.00	18.00	28 080.00	12.20	19 032.00	14.00	21 840.00	14.00	21 840.00
Rubble masonry	"	10 940	7.00	76 580.00	9.00	98 460.00	7.40	80 956.00	10.00	109 400.00	7.50	82 050.00
Dry rubble masonry	"	1 900	6.00	11 400.00	8.00	15 200.00	5.50	10 450.00	8.00	15 200.00	5.75	10 925.00
Excavation	"	10 000	2.00	20 000.00	1.00	10 000.00	0.30	3 000.00	0.40	4 000.00	0.50	5 000.00
Rock excavation	"	1 300	4.00	5 200.00	2.00	2 600.00	1.00	1 300.00	1.50	1 950.00	2.00	2 600.00
Embankment	"	5 000	0.50	2 500.00	0.75	3 750.00	0.30	1 500.00	0.30	1 500.00	0.50	2 500.00
Stone filling	"	2 000	1.25	2 500.00	1.25	2 500.00	1.25	2 500.00	1.25	2 500.00	1.00	2 000.00
Paving	"	1 200	2.50	3 000.00	2.25	2 700.00	1.50	1 800.00	4.00	4 800.00	2.50	3 000.00
Rip-rap (hand-placed)	"	1 500	2.00	3 000.00	1.75	2 625.00	1.25	1 875.00	1.25	1 875.00	2.00	3 000.00
Puddling	"	50	1.00	50.00	2.00	100.00	0.50	25.00	2.00	100.00	2.00	100.00
Bolt holes in masonry	Linear feet.	600	0.25	150.00	0.50	300.00	0.15	90.00	0.30	180.00	0.20	120.00
Miter-sills	Feet, B. M.	3 600	50.00	180.00	65.00	234.00	25.00	90.00	60.00	216.00	35.00	126.00
Framed timber	"	80 000	30.00	2 400.00	45.00	3 600.00	23.00	1 840.00	40.00	3 200.00	20.00	1 600.00
Sheathing	"	10 000	20.00	200.00	20.00	200.00	25.00	250.00	25.00	250.00	15.00	150.00
Grubbing and clearing	Complete.	....	......	2 400.00	......	300.00	......	1 000.00	......	50.00	......	200.00
Total	....	....	......	$179 200.00	......	$221 209.00	......	$163 348.00	......	$208 421.00	......	$177 331.00

The following items do not properly enter into the cost of the dam masonry:

Lumber and iron	$1 274.55
Carpenter work	689.77
Filling above dam	868.85
Cement	135.00
	$2 968.17

This gives a total cost of $13 368.66 for 6 130 cu. yds. of dry rubble dam masonry, or $2.180 + per cubic yard.

The bids received for Lock and Dam No. 4 are given in Table No. 7.

The work was let to the lowest bidders, at the prices given in bid No. 3.

The latest estimate of the total cost of this lock and dam is $204 500, itemized as follows:

Contract with lowest bidders	$166 260
Gates, valves and irons	12 400
Buildings	4 400
Engineering and incidentals	21 440
Total	$204 500

No detailed estimates of the cost of Locks and Dams Nos. 5 to 11, inclusive, have yet been made.

Commercial Importance.

The modern tendency in the United States seems to be for railroad transportation to supersede other forms of inland transportation. As a rule, the traffic on our rivers and canals is decreasing or at a standstill where well-managed railroads have come in competition, and the railroad traffic is steadily increasing. Also, the general tendency of railroad rates has been downward for many years, and the end is not yet.

There are several reasons for these conditions. Our rivers and canals, the vessels plying upon them, and the owners of these vessels, have, in many cases, not kept pace with modern improvements. They

are still trying to do business as they did fifty years ago. Our railroads are about the most progressive institutions we have, and they have in their employ much of the very best talent in the country. They are generally up to date, and aggressively fight water competition, at a loss, if necessary, and make it up elsewhere.

Much of our water transportation is stopped by ice for 4 or 5 months annually, while the railroads offer much quicker transportation regularly throughout the year, and have much better facilities for distributing freight to customers at terminals.

There are a few exceptions in which it seems almost impossible that railroads can ever compete with water transportation. Perhaps the most notable case is the coal traffic in barges from Pittsburg to New Orleans, a distance of 2 000 miles by river. Coal is said to be transported over this route at an actual cost of 50 cents per ton, or $\frac{1}{4}$ mill per ton-mile. The low cost of this traffic is due to good business management and the enormous quantities transported; 50 000 tons and more are frequently handled in a single tow from Louisville down. Probably no railroad would attempt to transport coal from Pittsburg to New Orleans for less than $3 per ton.

The Black Warrior River and its tributaries, the Mulberry and Locust Forks, split in twain the great Warrior Coal Basin, covering an area of some 3 000 sq. miles, and containing by far the finest and greatest deposits of coal south of West Virginia. It is only 400 miles by river from the heart of this coal basin to tide water at Mobile. The best and most extensive bodies of iron ore in Alabama, also the whole Birmingham District, with its wonderful mineral developments, lie within 12 to 30 miles by rail east of these streams. The lock system, when completed, will furnish water transportation throughout the year, for ice is practically unknown in this climate.

It is believed that mines along the river can place an excellent quality of steam coal, f. o. b. barge, at a cost of about 60 cents per short ton, and that the actual cost of transportation, when the lock system is completed, should not exceed 20 cents per short ton, or $\frac{1}{2}$ mill per ton-mile, a rate with which it is probable that railroad transportation can never compete. If these conditions should be realized, not only coal, but pig iron, steel and other mineral products of the Birmingham District can be placed in Mobile more cheaply than similar products can be placed in any other port in the world. This should

result in an enormous export trade from Mobile being rapidly built up, extending not only to the West Indies, Central and South America, but to the ports of all nations.

It is realized that the exact cost of barging coal through this lock system will never be known until it is done on a large scale, and then will vary from year to year, with the cost of labor, materials and incidentals; but there is no doubt that the cost will be much less than the present railroad rate of $1.10 per ton. Besides, the improved river will open a large area of coal lands, at present without railroad facilities and utterly inaccessible. Even on the supposition that the railroads will build into this territory and handle the traffic, with an improved river ready to compete with them at all times, they would have to do it at a much lower rate than $1.10 per ton, and the public would save the difference. In fact, one of the strongest arguments in favor of the improvement of rivers and canals by the General Government is that they serve to regulate and control railroad rates more effectively than legislation ever can. Able railroad attorneys can generally find some means of evading inimical regulations framed by legislative bodies. Well-managed railroads will probably always find some way of pooling interests when it is greatly to their advantage to do so. But it is a difficult matter to pool with a public waterway, operated and maintained by the General Government without tolls, because any town or individual with a few thousand dollars can build an independent boat and "break the combination."

For these reasons, the writer firmly believes that the improvement of these rivers is *pro bono publico*, and therefore worthy of being done by the General Government.

DISCUSSION.

Mr. Sibert. WILLIAM L. SIBERT, M. Am. Soc. C. E. (by letter).—Mr. McCalla's paper presents in detail a subject about which there is very little literature. The conclusions, however, which have found expression in the plans presented in the paper, are very different in some important particulars from the conclusions to which the writer's experience has led him.

These particulars will be the subject of this discussion.

Lock Location—Guide and Guard Walls.—On this subject Mr. McCalla states as follows:

"Locations were sought in wide, shallow places with good, high banks, and in curved instead of straight reaches. The locks are always located on the convex shore, in order to secure better protection from drift during floods, and on straight approaches parallel to the axis of the lock re-entering the stream."

The proposed guide and guard arrangements for entrance and exit are as follows: A paved slope, 2 ft. on 1 ft., above the lock, and a line of pile clusters below. The guard arrangement above the lock is a line of pile clusters flaring toward the middle of the stream, with nothing below. See Fig. 16.

The writer's experience leads to the following:

A lock with a fixed dam should be located, if practicable, in a wide straight reach of the river. If forced to locate a lock and dam in a bend, by foundation or other conditions, the lock should be placed on the concave side of the bend. There should be a guide wall, having the inside face vertical, on the land side of the entrance. The alignment of this wall should be such that boats alongside of it can enter the lock without material change of direction. The least length should be such that boats can come alongside of it before the head of the craft comes into the dead water at the entrance. The maximum length should depend on the rapidity with which it is necessary to operate the lock. Where a lock is taxed to do the business, the guide wall should be long enough to allow boats to come alongside and be in position to enter the lock, so that no delay will result when the lock is ready for a boat. Mooring piers, for boats awaiting lockage, are necessary, and guide walls, when long enough, form the most convenient piers.

The guard wall should be solid, with possibly a short drift gap next to the head of the river lock wall, and should be of such length that one lockage lying inside of it will be safe if cut loose from the steamboat. The solid wall will deaden the water in the entrance so that barges can be taken into the lock by hand. This wall should flare slightly toward the middle of the stream, but not enough to create an appreciable eddy below the dam near the lower entrance to

the lock. This flare should be prolonged by pile clusters. A paved slope immediately above the lock is objectionable, for the reason that the wheels of boats will be damaged by striking upon it. Mr. Sibert.

A guard wall made entirely of clusters of piles from 30 to 70 ft. apart, makes the entrance to the lock very difficult and dangerous. When the water is high, the draft of the water between the pile clusters pulls everything hard against the clusters, making it impracticable to place barges in the lock by hand. The forward outside corner of a barge is likely to be drawn in behind a cluster, and a steamboat would be forced to put out a line to get the barge out.

The guide wall below the lock should be solid, with the inside face vertical. Its height should be such that it will be flooded at the same time that the upper end of the lock wall is flooded; its length to be determined in the same way that the length of the upper guide wall was determined.

Guard walls on the lower end of the river lock walls should be avoided where possible. Where the dam is not located near the head of the lock, and when the lock is short, the reaction sometimes forces the construction of such a wall. It is absolutely necessary that gates be mitered properly, and when there is any danger that the reaction below the dam will disturb this mitering before enough head is on the gates to hold them together, a guard wall should be built. In the writer's opinion, some of the recent failures of wooden lock gates are due to this cause. This reaction disturbs the miter after there is some head on the gates, and drives one of the leaves up stream, as shown in Fig. 21. The pressure on the upper side of the other leaf forces the upper part of that leaf down stream, so as to come in contact with the miter-post of the opposite leaf, as shown. As the water rises in the pit the leaves are held in that position, and one leaf is forced by the other when the head is sufficient. A guard wall below the lock, with the guide wall, forms a pocket which is always full of drift during a rise, which, of course, is a nuisance.

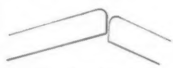


FIG. 21

In order to discuss the question of lock location in a bend, assume that one lock is located on the convex side and one on the concave side. The lock located on the convex side, Fig. 22, has the guide and guard arrangements proposed in Mr. McCalla's paper; that located on the concave side, Fig. 23, has such guard and guide arrangements as are thought necessary.

Let it be assumed that a towboat navigation is to be especially provided for, or such as will predominate on the Warrior system. The various positions of a tow entering the lock for both locations are shown on Figs. 22 and 23.

It is seen that a tow in entering the lock on the convex side is forced to keep the stern of the towboat next to that side, for if the

Mr. Sibert. stern gets out into the stronger current it is difficult to prevent the tow from rounding to. In high water, when boats have their full capacity of tow, this can only be done safely by the use of a line. Assuming, however, that it can be done without a line; when the boat and tow are in the position shown at *A*, it is then necessary to back the stern of the boat out, thus swinging the tow in the entrance. Such an operation in a swift current just above a dam is exceedingly dangerous, and, if attempted without the use of a line, will result in many tows going over the dam.

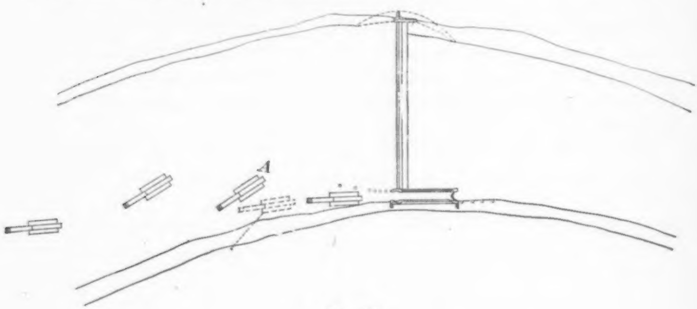


FIG. 22.

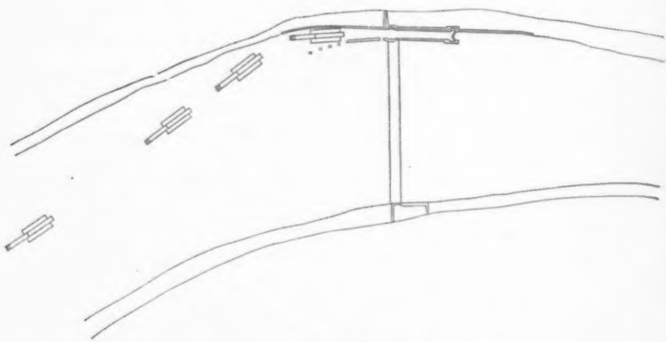


FIG. 23.

It is also difficult to enter the lock on the convex shore in low water. When the tow is in the position *A* it is necessary to throw the rudders away from the shore in backing to get the tow in shape to enter. With a stern-wheel boat, this soon gives rise to a down-stream current on the land side of the tow, due to the rudders deflecting the current made by the wheel. This current is again deflected by the bank and dead water at the head of the entrance, and it has a tendency to

push the head of the tow outside of the entrance. The stern of the boat is already swinging out into the river, with the result that the entire tow gets into such a position as to force a beginning over again of the operation of entering the lock. Mr. Sibert.

Now, upon examination of the positions of the tow entering the lock on the concave side, it is seen that no lines are necessary, and no unusual difficulties presented. The current naturally takes the boat and tow in toward the shore. It may be necessary to back the stern out to keep off the shore, but in this operation the rudder is thrown toward the shore and the extra current generated is thrown on the river side of the boat and tow.

It may be more difficult to get out of the lower entrance of a lock located on the concave side, but there is no danger attached to it. Safe navigation should be the first consideration in lock location.

Regardless of the safety of navigation, the location of a lock on the concave side of a bend is still preferable to the convex side. The convex side is always shallow. The deposit naturally made on this side can be dredged away, but the same conditions that placed it there in the first place will replace it when removed. Of the two evils, drift or the silent process of deposit of silt during rises, the former is more easily dealt with. An alert lock force can steer all drift over the dam. The dead water in the head of the lock will facilitate it, but the silting up of the entrances cannot be stopped, and, after a rise, a dredge will often be necessary before a lock can be put into service.

A straight wide reach of river is free from the disadvantages of a bend location with the lock on either side. A lock should be so located, and the dam made of such height, that no dredging will be necessary in the upper ends of the pools in order to produce the requisite depths. Such dredging is a continual expense. The resulting channels are always narrow, and interfere with the full use of the river for towboat navigation. The greatest amount of deposit is produced in the upper ends of the pools. The slackening up of the current below the dams on a rising river, until the water piles up enough to create a slope sufficient to carry the discharge, causes a deposit of heavy material in the reaches just below the dams. This makes it evident that the full depth should be made in the beginning.

It has been the history of all improvements that, as soon as the improvement is completed, the navigation interests begin to clamor for more depth, as the profits of transportation vary directly with the depth. One way of giving this depth is by dredging in the upper end of the pool, and, in view of such a contingency being forced by legislative action, it is always well to make the pool depths by the dams in the beginning, and to place the sills below the projected navigable depth.

Dams.—The decision to place fixed instead of movable dams in the Warrior River, in the writer's opinion, was wise. The disadvantages

Mr. Sibert. of movable dams are many. They are frail structures; the likelihood of serious accident on account of drift and ice has rendered it necessary to keep the Davis Island Dam, on the Ohio River, down for at least six months in the year, from December to May, inclusive. During this time there is often low water, with the result that there is not enough water for the transaction of the river business in the pool above the dam, which includes the Harbor of Pittsburg. When it is remembered that a line of transportation must be in operation practically all the time, in order to develop its full use under modern conditions, the almost fatal defect of a movable-dam system seems apparent, and its use should be limited to particular cases. Such cases should be: Rivers in which there is nearly always sufficient water for the business during the winter and spring seasons, unless some more efficient system of movable dams can be devised. Such a system should be one that can be surely lowered before the drift or ice of a sudden rise can catch it, and one that can be raised and maintained notwithstanding such drift or ice as may be running when the stage of water is reduced below that needed for the most economical transportation of commodities.

In the upper reaches of all rivers where quick rises occur, fixed dams with long locks, as wide as practicable, seem to be indicated as the best system. In the lower and flatter reaches movable dams may be the best.

Types of Fixed Dams.—Figs. 24 to 28 are cross-sections of the dam proposed in the paper, and of dams which the writer considers good types. There is also shown a cross-section of the type of the first fixed timber dams built in this country.

Generally speaking, the less upper slope on a dam, the less leakage area; it being impracticable to maintain an efficient backing on such slope. In the earlier types of dams, the upper slope extended from the crest to the river bottom, the idea being to make the water pressure increase the stability of the dam. Leakage was abundant, of course, and such dams could only be maintained on hard rock bottoms.

A gradual change from this type to those generally adopted now is seen in the various dams built for slack-water purposes in this country in the last sixty years, the length of the upper slope decreasing and of the lower slope increasing.

An examination of the section proposed by Mr McCalla shows, as far as it can be seen, that every possible attempt has been made to box in any leakage that may find its way through the upper face of the dam. The lower slope is double-lapped and caulked, triple-lapped sheet-piling is driven along the down-stream edge of the dam proper, the lower breast is sheeted, the sheeting on the apron is double-lapped, and triple-lapped sheet-piling is driven along the lower edge of the apron.

Mr. Sibert.

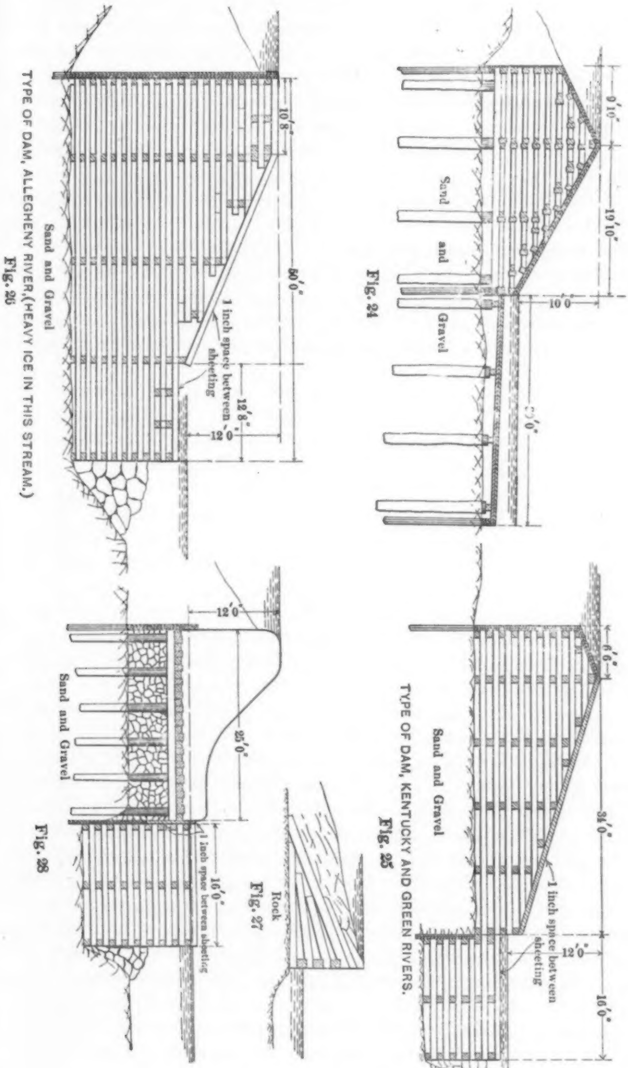


Fig. 24

TYPE OF DAM, KENTUCKY AND GREEN RIVERS.

Fig. 25

TYPE OF DAM, ALLEGHENY RIVER, (HEAVY ICE IN THIS STREAM.)

Fig. 26

Fig. 27

Fig. 28

Mr. Sibert. In the foundation for the lock, Mr. McCalla's plan wisely provides for relieving the floor from any pressure from the upper pool due to leakage through the triple-lapped sheet-piling; why the same principle was not applied in the dam design, is not seen, when it is known that it is practically impossible to make the filling of crib dams absolutely impervious to water. The sheet-piling at the lower edge of the dam and apron, in the plan presented, is an acknowledgment of this fact.

In the sections showing types of dams on the Green, Kentucky, and Allegheny Rivers, the dam is made by the upper breast and slope. The remainder of the structure is for the purpose of holding the upper breast and slope to their work, and for controlling the flow of the water over the dam in such a way as to protect the structure itself from the action of the water under the imposed conditions of flow; also, to reduce the disturbance below the dam, for reasons affecting navigation.

The sheeting on the lower slopes of these dams is of one thickness of timber, placed so that there will be a 1-in. opening between the pieces of sheeting at the lower end of the slope. This opening may be gradually decreased to nothing at the comb of the dam.

The lower breasts of the dams and aprons are of open crib work, and the pieces of sheeting on the apron are spaced 1 in. apart; the idea being to let out any leakage that gets into the body of the dam.

The writer has seen two cases where the sheeting (not double-lapped, but laid close on the aprons) was taken off; and several cases of lifting of the lower slopes of dams, due to close-fitting sheeting.

Assuming a leakage through the dam and under the apron, as proposed in the paper, a simple calculation will show that the pressure to be resisted by each holding-down driftbolt in the apron (there being one in each pile, 9 ft. each way) will be excessive.

No reason is seen for a pile foundation for a timber crib dam, unless it be the cost. If a concrete dam, or an ultimate rebuilding in concrete, is contemplated, there is a reason, and a section for such a dam is shown in Fig. 28. In the design of this dam, the reduction of the cost of the coffer-dam to a minimum was aimed at, and it is thought that such a dam can be built at low cost. The timber foundation for the concrete top can be built without the use of a coffer. The concrete base, a section at a time, as high as low water, can then be put in behind the bulkhead made by the projecting sheet-piling. In building the remainder of the dam, a sufficient number of openings (each about 10 ft. wide) to carry the low-water discharge should be left. With this arrangement, no head will be produced at the dam until the closing of the 10-ft. openings is commenced. Beams across these

openings having been placed while building the contiguous sections, Mr Sibert. little trouble should be experienced in bulkheading off these openings so as to fill them with concrete.

Length and Lift of Dams, and Height of Lock Walls above Crest of Dams.—This is a subject about which there has been very little theoretical discussion, as far as the writer knows. The available data for such a discussion are very meager.

The result desired is to apportion the foregoing elements of lock design so that, when the lock walls are submerged, navigation over the dam is practicable.

It seems to have been solved in the paper by adopting a uniform lift, uniform height of lock wall above the crest of the dam, and a uniform length of dam. Apparently, the length of the pools above and below the dams, which depended of course upon the slope of the unimproved river, was left out of consideration. This element is a very important one.

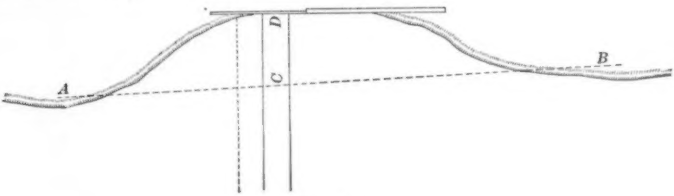


FIG. 29.

The lift and height of the guard wall at Lock No. 1, Black Warrior River, being such as to practically produce the required condition, a similar lift and height of guard wall was adopted for the Warrior system.

An examination of Fig. 1 shows that the length of the pool below Lock No. 1, Black Warrior, is 46 miles; the first five pools in the Warrior River vary from 15 to 21 miles in length. In view of the fact that the fall at the dam is due, not only to the height of the water surface at the crest of the dam, but to the elevation of the water surface just below the dam, which latter condition is determined by the slope necessary for the discharge of the flood through the pool below, it seems to follow that, since a greater elevation of water at the upper end of a long pool is necessary to produce a certain slope than is necessary in a short one, boats cannot ordinarily go over the dams in the Warrior River when the lock walls are just flooded.

The effect of lengthening the dam by digging away the bank locally, as shown in Fig. 16, can very easily be over-estimated.

Dam No. 1, on the Monongahela River, Fig. 29, is now of about the same design as proposed in the paper. This condition was pro-

Mr. Sibert duced by the unauthorized building out of the banks above and below the dam. The question is: What difference would it make if the bank was filled out to the line $A - B$? It is noticed that during flood stages the fall at the dam is considerably less from C to D than it is along the remainder of the crest. Boats often go over the dam from C to D, when they cannot go over the other part. This seems to show that the effect of this local widening is not felt entirely across the river or for any distance above. The effect of such a widening will

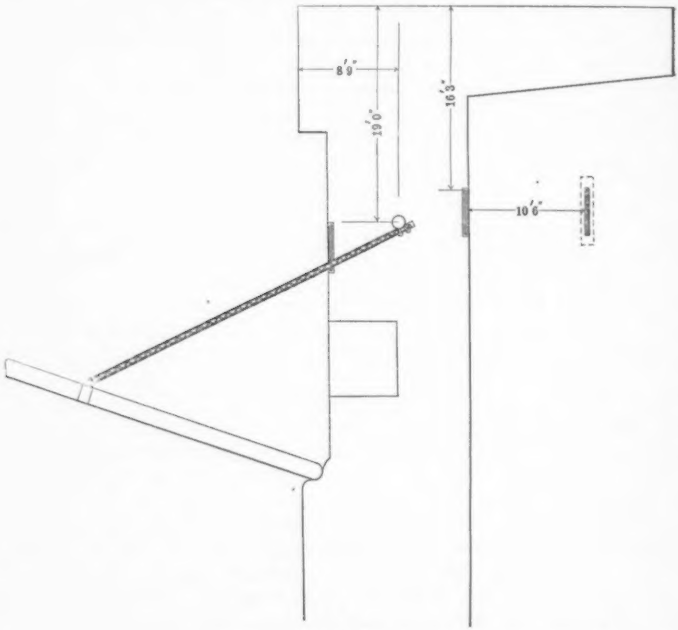


FIG. 30.

vary somewhere between that apparently expected and that produced by filling once the extra space. To produce the expected result, the channel of the river above should be gradually widened to the dam. The probable effect of the sudden narrowing of the channel below the dam, in the manner indicated in Fig. 16, will be that the water will be deflected toward the opposite shore and increase the violence of the eddy that will exist below the lock. This eddy passing through the pile clusters will cause difficulty in entering the lock from below.

There are not many points in the construction of the lock itself Mr. Sibert. that vary materially from the practice with which the writer is accustomed. Some few are worthy of mention, however.

On the Monongahela River, in building concrete locks, work on the 20 to 30-ft blocks or monoliths is never stopped, night or day, until the monolith is completed.

Scour below a dam is greatest next to the lock wall and abutment. Sometimes a fill will be noticed in the center of the stream, while a deep narrow channel will be made along the lock wall, due probably to the guiding effect of the wall. In some instances on the Monongahela River this cut is 25 ft. deep. This would indicate that the penetration of the sheet-piling to a depth of 12 ft., as shown in the plan, was not sufficient, unless a submerged protection crib was built alongside and outside the lock wall.

The method of opening the gates is a departure from the usual practice, and an undesirable one, it is thought. It has the appearance of taking hold of the wrong end of the gate. Of course, it is possible to apportion the lever arm and force in such a way as to overcome the calculated resistance, but there is often an uncalculated resistance, due to the jamming together of the miter-posts, especially when these posts are somewhat old, that is difficult to overcome by a force applied near the resistance. In starting gates thus jammed, on the Monongahela River, $\frac{3}{4}$-in. chains have been broken, the point of application having been near the toe, and not at the heel. The usual methods, *viz.*, by a rack spar working in a pinion on a capstan, Fig. 30, or by a spar and rope seem preferable in medium and small-sized locks. In the larger locks, opening and closing the gates by chains is the usual practice.

D. A. Watt, M. Am. Soc. C. E. (by letter).—This paper will prove Mr. D. A. Watt. of much interest to engineers connected with works of canalization, because, outside of the reports of the Chief of Engineers of the U. S. Army, very little information on the subject is accessible.

One feature of the foundations is worthy of note, namely, the omission of a timber floor on the piles under the main walls. This is a practice which is not often met with in the design of locks, the usual method having been to drift-bolt caps to the heads of the piles, and spike on a plank flooring on which a concrete foundation course is commenced. By this method, however, the entire weight of the masonry has to be carried by the piles, and the additional support which could have been obtained from the natural foundation, if the concrete had surrounded the pile heads, is not turned to use. Besides this, the strength of the work depends on the ability to resist crushing of those parts of the caps which rest on the piles, and through which all the load must pass. While these may be strong enough when dry, they become weaker when wet, as experience has shown that timbers

Mr. D. A. Watt. long submerged and under pressure become softened by the water. It is probable that the cracks, which have appeared after a lapse of years, in some walls built on pile foundations, have been due to this cause.

The design of the floors of locks is sometimes open to similar criticism. These portions, which are so vital a point in the structure, too often bear the stamp of lack of permanency, whereas, in point of fact, they ought to be the last parts ever to need repairs. The opinion of General William P. Craighill, Hon. M. Am. Soc. C. E., ex-Chief of Engineers, is well worthy of note in this respect, when, in a report, he stated that a long experience had convinced him that in all construction connected with a lock, especially where the portions would be submerged, nothing but the best and most permanent work was worth putting in. This fact has been long known to European engineers, and the use of impermanent materials, such as timber floors, for permanent construction, was long ago discontinued by them. They are well acquainted with the fact that timber, to be permanent under water, must be protected from all currents, or it will be worn away, slowly if exposed to a slight current, and rapidly if exposed to a strong one. It is true that on a lock floor the fibers will be eroded slowly and at a very different rate from the sheathing of a dam, but the effect will be none the less sure, and, as a renewal will mean a stoppage of navigation, pumping out the lock pit, and, in extreme cases, having to build a coffer around the entire lock, the amount saved by a cheap first construction would appear to be a very questionable economy. For a small extra expense, where concrete is available, a solid arched floor could be put in, one that would be free from erosion and free also from that worst of failures in a lock, an upspringing from the pressure below. In the Suresnes lock, just below Paris, completed in 1885, the floor consists of an arch of cut stone, with a rise of 20 ins. in the chamber width of 59 ft., and laid on a bed of masonry several feet thick. This is a most durable method of construction, and, while it may not be followed to the full extent in America, it may be hoped that construction which is impermanent, or economical only in its first cost, will soon become a thing of the past.

As regards the underdraining of a floor, which appears to be a source of anxiety to many engineers, the writer's experience has led him to believe that the only safe way to dispose of the danger from upward pressure is to build the floor strong enough to resist it, and to let the seepage from the upper pool force its way out by making a natural channel of its own. For this purpose there is no section better adapted than the inverted arch, because, for the amount of material, it is by far the strongest type of floor built. The method of providing stone sub-drains seems to be of questionable utility. The

rip-rap filling in a crib, in rivers of ordinary sediment, will become completely choked with mud in a single winter, and, as such cribs are usually exposed to more or less current, it would seem reasonable to believe that a sub-drain exposed in time of floods to no passage of water would choke up as readily, and thus defeat its purpose. Mr. D. A. Watt.

The proposed relief by using ball valves, as shown in Fig. 8, while novel and ingenious, is open to objection in that they will render the pumping out of the lock very difficult. Their use is based on the theory that the sheet-piling will prevent seepage from the outside, except to an extent which can be cared for by the pumps. The writer had occasion to pump out a chamber some time ago under a lower-pool head of about 6 ft., the lock in question being on a gravelly foundation, and surrounded by a line of sheet-piling driven as well and as carefully as could have been done in work of this class. The floor was of piles and double plank, overlaid with concrete. During the repairs, some holes had to be drilled to the plank, and wherever one came in contact with a joint a strong jet of water came up through the hole. Moreover, when pumping was begun (with a 10-in. centrifugal pump), it was found impossible to lower the water more than about 1 ft. Investigation showed that the stone drain usually placed behind a land wall had been arranged to discharge through a 12-in. pipe into the tail-bay, so that it had no communication with the pool when the lower coffer was in, and the water flowing through it must have come under the foundations from outside. The pipe was stopped up with a wooden plug, and the chamber emptied without further difficulty. From such evidence of the passage of water beneath a porous foundation, it would appear reasonable to believe that where openings are made through a floor it will be a matter of considerable difficulty to pump out.

George Y. Wisner, M. Am. Soc. C. E. (by letter).—The engineering features and details of the lock construction on the Black Warrior, Warrior and Tombigbee Rivers, described by Mr. McCalla, form a valuable contribution to the literature on river improvement in the United States. Like several other large improvement projects now being executed by the General Government, the question of whether the benefits to be derived will warrant the expenditure involved has been given but little consideration, and the probable cost of transportation per ton, as stated, is by no means a fair one for comparison with railroad rates between the same terminals. Mr. Wisner.

An examination of the map of Alabama shows fairly good railroad facilities between the coal fields and the Gulf ports, and the rate quoted, $1.10 per ton for a distance of upwards of 400 miles by water route and 275 miles by rail (about 4 mills per ton-mile by rail, and 2.7 mills per ton-mile for distance by the water route), is certainly not a heavy tax on the development of the mining industries, and is

Mr. Wisner. probably as low as the freight can be carried profitably by rail at present, and nearly as low as the rate would be on the completed waterway if all fixed charges, due to construction, maintenance and operation, were included in the rates.

If, in order to promote competition and secure minimum railroad rates from the coal fields of Alabama to the seaboard, the General Government should furnish a right of way, grade the roadbed, lay the rails, and maintain a first-class double-track railroad between the freight terminals at the mines and seaboard, the existing railroads would have just cause of complaint. Yet, to construct and maintain a waterway between the same terminals, free of fixed charges for construction, maintenance and operation, involves exactly the same objections. If the sole object of constructing and maintaining a free waterway is to establish transportation rates, to meet which existing railroads must be operated at a loss, there are certainly grave doubts whether the work is justified or warranted.

In the writer's opinion, the only conditions which warrant the General Government in constructing a free waterway, between terminals where first-class railroad facilities already exist, is where such waterway will develop new commerce, which otherwise could never be made profitable, and which will indirectly build up new freight traffic of benefit to the railroads as well as to the waterway.

The result of the improvement of the waterways of the Great Lakes is a striking example of building up new commerce which otherwise could not have been established, and which indirectly has resulted in a railroad traffic between the Lake ports and the seaboard which has made the railroads, in competition with the Lake waterway, the best dividend-paying roads of the country.

The Illinois and Mississippi Canal, from the Mississippi River near Rock Island to Lake Michigan at Chicago, is an equally striking example of useless waste of public money for a waterway which will never develop new commerce to any material extent, nor control the rates on railroads between terminals. Some interesting and instructive engineering work in the way of concrete lock construction has been done on this canal, and, as an example of how to build good concrete locks, the work is of great value to the profession, but, from the point of view of whether it will pay, either directly or indirectly, the project should never have been undertaken.

Under which of these classes of improvements the Alabama work falls is a question which should have been more thoroughly considered by the Government officials than would seem to have been the case.

The author states that "the actual cost of transportation, when the lock system is completed, should not exceed 20 cents per short ton, or ½ mill per ton-mile." This rate does not include anything for interest on the cost of construction, annual expenses for maintenance,

repairs and operation, or for profit to the transportation and transfer companies. Mr. Wisner.

The amount by which these fixed charges will increase the transportation rate cannot be stated definitely without a knowledge of the approximate volume of traffic to be expected.

On the Erie Canal, a waterway of approximately the same depth as the one under consideration, the transportation rate averages about 2 mills per ton-mile, and as both are free waterways it is fair to presume that the rates on the Alabama rivers will not be much different from that of the Erie Canal.

On the Chesapeake and Ohio Railroad, it is stated that coal is carried at a profit on a rate of 2¼ mills per ton-mile, and since this rate involves fixed charges for cost of construction and maintenance of the road, it is really comparatively less than the Erie Canal rate.

The Bessemer and Lake Erie Railroad is said to carry ore from Conneaut to Pittsburg at an actual cost, for transportation alone, of 1½ mills per ton-mile, the actual freight rate being 3.65 mills per ton-mile.

If the business between the coal fields of Alabama and the seaboard will warrant the expenditure of $5 000 000 for a free waterway, there is no reason why a double-track freight railroad, capable of handling coal as cheaply as the Chesapeake and Ohio Railroad, could not be operated at a profit on practically as low a transportation rate as is likely to be charged on the improved waterway, if the fixed charges due to cost, maintenance and operation be included.

In all estimates of transportation rates, it should be noted that there is a vast difference between the actual cost of moving a ton one mile in an open waterway on a loaded vessel, and the rate which must be charged to cover profits, deterioration of works and vessels, repairs, operation, maintenance, fixed charges, and loss from terminal and other detentions. It is extremely doubtful if the boats of the river waterway would be able to go sufficiently near the mines to be loaded without first handling the coal with cars over several miles of railroad, which would involve extensive transfer sheds, and a large addition to the cost of transportation on the waterway, which would be eliminated from the rate if carried on cars direct from the mines to the seaboard.

No data are given in the paper on which estimates for the annual cost of maintenance and operation can be based, but, if the waterway is to be a commercial success when completed, it will be necessary to operate it 24 hours each day. This will require three shifts of men at each lock, and as the rivers are very crooked and narrow—in many places being only 100 ft. wide—a thorough system of lighting for both locks and channel will be necessary.

For the present consideration, the following approximate estimate of the probable annual expense account is based upon a volume of

Mr Wisner. traffic requiring a daily 24 hours' service and the waterway maintained in good condition:

Annual cost of operation and supplies: Twenty locks at $10 000	$200 000
Lighting of locks and channel	50 000
Maintenance of locks and channel	50 000
Extraordinary repairs, supervision and contingencies	50 000
Interest on $5 000 000 at 3%	150 000
Probable annual expenses	$500 000

It is apparent, therefore, that, until the volume of traffic exceeds 1 000 000 tons per year, the cost of transportation due to the necessary fixed charges alone will be 50 cents per ton, or more; to which must be added a transportation rate of probably 1½ mills per ton-mile, making the actual cost per ton between terminals practically the same as the present railroad rate.

It is true that with a free waterway the fixed charges, amounting to about one-half the total annual cost of transportation, will be assumed by the General Government, but, as there are no public lands to be developed in the coal regions of Alabama, from which the general public are to receive any returns, it would appear that the only parties to be benefited by the improvement are the owners of the coal and iron mines of Central Alabama. If this is the case it is an open question whether the improvement is of sufficient national importance to warrant the General Government in assuming one-half the cost of transportation of the products from the mines to the seaboard, and whether, if the project is sufficiently meritorious to warrant the expenditure, it would not be more just to the people of the entire country to charge sufficient toll to make the enterprise self-supporting, and thereby place the annual expenses on the property and business to be benefited.

It is not the writer's purpose to question the advisability of the project, which, from the data available in the reports of the Chief of Engineers, cannot be stated definitely, but to raise the issue as to who should pay for and maintain improvements of waterways and harbors where the benefits are purely local and have no direct bearing on the commerce of the whole country. The large number of projects provided for in the River and Harbor Bill of 1901 which were open to this criticism caused its failure to pass Congress, and the bill which recently became a law narrowly escaped the same fate, for similar reasons.

There is a tendency on the part of legislators to scrutinize this class of expenditures more closely than heretofore, and engineers having charge of projects requiring national support will have to show wherein the commerce of the country will be either directly or indirectly benefited, or the necessary appropriations will not be obtained easily in the future.

JOHN M. G. WATT, M. Am. Soc. C. E. (by letter).—The problems involved in the design of a lock to be constructed on the solid rock are neither difficult nor unusual, being simply those of a retaining wall, or of a wall to resist hydrostatic pressure, combined, in certain parts, with the thrust from the gates. The chief points to be considered are the details, principally those of the filling and emptying valves and the machinery of the lock. If, however, the foundations are not on such favorable material, complications set in, and these must be provided for as each particular case may demand. The writer does not propose to discuss the general principles of lock design, but to call attention to some details which have proved useful in his experience. Mr. J. M. G. Watt.

The guard of a lock with a fixed dam, or the height of its top above the dam crest, must be determined in accordance with the varying conditions. Theoretically, it should be such that navigation over the dam can be had from the moment when the lock passes out of use through submergence. Practically, other conditions may determine this. To illustrate: At Lock No. 8, on the Kentucky River, where the guard is 10 ft., the fall over the dam, when the lock passes out of use, is from 7 to 9 ft., consequently there can be no passage either through the lock or over the dam. To render the lock navigable until the river rises to such a height as to obliterate this fall, the walls would need to be raised about 5 ft., necessitating a comparatively large outlay for the incommensurate result of a few extra days of navigation each year; but, at this stage of the river, the current is very swift, and very heavy drift is generally running; besides this, many of the regular landings are submerged, and passage under some of the bridges over the river is impossible; therefore navigation is suspended until the rise has partially subsided. In view of this condition it would not have been economy to make the guard 15 ft., and it was therefore made such that the lock could be used up to the stage when boats generally stopped running.

For filling and emptying the lock, in the writer's opinion, culverts in the walls, passing around the hollow quoin, are the best. Valves in the gates are a possible source of weakness, and, whether opened with worm-gear or with rack and quadrant, all offer the objection that they have to be maneuvered on the narrow width of the gate, in all sorts of weather and temperature, and both day and night, being especially dangerous when the gates are coated with snow or sleet. Culverts in the walls may be closed with butterfly valves on a vertical axis, or with cylindrical valves. If the former, the frames in which they fit should be of metal, not of wood, and should be cemented water-tight when set. Those with which the writer has had to do are made, some of cast iron, others of wrought plates, channels and angles, all with unequal arms, and opening with the longer arm inward and up stream. These arms are designed so that, when the valve is closed, the longer

Mr. J. M. G. Watt.

arm has less of its surface exposed to the water pressure than the shorter, and, consequently, the excess of pressure keeps the valve closed. Cylindrical, cast-iron valves are in use upon the Muskingum River, and have given satisfaction; they will be used in Locks Nos. 1 and 2, on the Big Sandy River, Locks Nos. 9 and 10, on the Kentucky River, and in the proposed lock at Colbert Shoals, in the Tennessee River. They have the advantage of simplicity and of ease in operation, and are set farther back from the face of the wall than the butterfly valves, thus giving the lockmen greater safety when operating in icy weather.

For opening and closing the gates by hand power, the writer, after much investigation, has found nothing entirely satisfactory, but the best, in his opinion, is the steel rack-bar, built up of two channels, latticed, with cast-iron rack, attached to the top of the gate at about one-third the distance from the miter-post, or less. This rack is operated by a gear wheel on which is a capstan head with sockets for bars, and which can be set back far enough on the wall to ensure safety to the men if they should slip when nearest to the lock pit. For the river walls, this rack should be supported, when the gate is open, on a bracket on the outside of the wall, removable in times of flood.

For coffering the upper and lower bays, in order to make repairs in the chamber, permanent trestles to support needles have been used. The writer considers these objectionable, for two reasons: First, the danger of deterioration by rust. These trestles are constantly under water, and inspection cannot be had, nor can they be repainted without removal by a diver, or unless the pool is drawn, or another coffer put in, suspending navigation for the time; consequently, when the time comes to use the trestles, they may prove unreliable because of loss of strength by rust. The other reason is that they are in the way of dredging. Trestles were put in the upper entrance to Lock No. 6, on the Kentucky River, when built in 1891, but there has never yet been occasion to use them. Dredging is necessary there every year after the winter and spring rises, and great care is exercised by the dredge runner at that spot, yet, in spite of this care, the trestles are struck more or less often by the dredge dipper, and one was caught so badly that it was torn out; whether the others are in a fit state to use, or whether they are bent or distorted, is not known. For locks having a width of not more than 52 ft. in the chamber, timber beams can be built up to span the opening, letting the needles rest against them and against a shoulder made for that purpose on the lock floor. For larger spans it would be desirable to use a removable steel truss above the water level, supported on temporary timbers, or removable trestles, both with steel needles formed of I-beams and buckle plates, from 2 to 4 ft. wide, or with ordinary timber needles. The journals for these trestles would be set permanently, and would be made extra heavy to resist rust. The trestles would be 10 ft. apart, and it would take but

a short while to set them and their needles in place. After use they would be removed, cleaned and taken to their place of storage, where they could always be inspected and kept in good condition. Mr. J. M. G. Watt.

The question of whether a fixed or a movable dam should be used, is one that must be decided by the requirements of the commerce of the river under consideration.

For a fixed dam, the experience of the writer has led him strongly to prefer a concrete to a timber structure, as being more economical, and less a cause of worry. Timber is supposed to last indefinitely under water, and, as a general rule, it does, but it is just the exceptions that cause all the trouble, and the writer has been compelled to tear out many thousands of feet of good timber in order to get down

CONCRETE DAM NO. 9, KENTUCKY RIVER, KY.

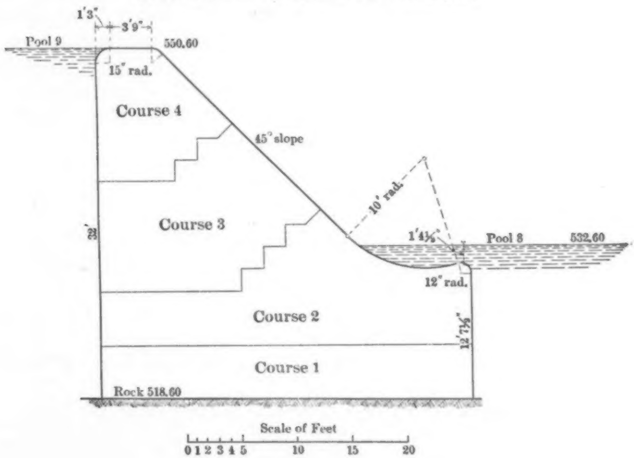


FIG. 31.

deep enough to that which had rotted and was causing the overturning of the structure. When the cost of this demolition and reconstruction is taken into account, it will be found that concrete is just as cheap. If concrete is used, the down-stream slope of the dam should not be steeper than 45°, and it should terminate in an apron with a flat curve upward. This apron should be as long as possible, in order to give a good discharge to the water. See Fig. 31.

If a dam of timber cribs filled with stone is used, it can, unless the river is a very swift one, be built along the shore, in quiet water, for several courses, and then be towed to position, sunk and filled. The bottom course of timber should be blocked up so as to fit the bed of

Mr. J. M. G. Watt. the river, approximately, according to soundings taken before building; this has been carried out successfully by the writer with cribs 250 ft. long, 50 ft. wide and 8 to 10 ft. high, with very little trouble. This method has the advantage of leaving the river open until the last moment. This kind of dam is dependent only upon its weight, the timber cribs being simply so many boxes to prevent the stone from being washed away, and all dapping of cross-timbers and splicing of longitudinal timbers is an unnecessary expense. It has been the writer's practice to use butt joints everywhere. In the face of the work the butt joints of the stringers come over a tie; in the interior they may come over a tie or anywhere between two ties; the result has been quite satisfactory and cheap. The use of timbers square in cross-section, and not simply oblong, is also advisable, for economy. Any side may then be put uppermost, and no time is wasted in making extra turns. This item is small, but is worth consideration. The most economical size of timber on the work on the Kentucky River has proved to be 10 x 10 ins., this being more easily obtained than 12 x 12 ins., and being amply large. The pens of dams and cribs are all 10 ft. from center to center each way. The sheathing on the up-stream side of the dams is formed of one layer of 3-in. plank, covered by 2-in. plank, breaking joints. These can be laid close enough to obviate any necessity for caulking, but should any joints open they will soon be filled with deposit. The top layer of the sheathing is in two lengths, the one next the crest being short, as it can then be removed easily when worn out, and replaced without drawing the pool down more than 12 or 15 ins. Probably a still better way to construct the crest would be to stop the sheathing 12 ins. from the crest, and fill this space with longitudinal 12-in. plank. This would entail less labor and time in tearing up when worn out.

Mr. Nelles. George T. Nelles, M. Am. Soc. C. E. (by letter).—During a connection of six years with the improvement of the Tennessee River and its principal tributaries, the writer had occasion to make a very comprehensive study of the improvement of non-tidal rivers in general, and to design and execute works of improvement very similar in their character and object to those described by Mr. McCalla; he, therefore, takes great pleasure in discussing the paper, and extends his discussion to the question of lock designs in general.

Method.—The improvement of rivers of this class can generally be effected either by regulation or canalization. The former method comprises the removal of obstructions, the rectification of the channel and the equalization of the slope and depth by means of excavation, training and contracting works. A review of many existing examples shows that the success of regulation works, where the flow is ample and the natural conditions not incompatible, depends upon their design and execution in accordance with correct hydrotechnic prin-

ciples, upon the stability of the bed and bank, whether natural or produced by artificial means, and upon the conservation to the greatest possible extent of the natural shape, conditions and tendencies of the stream. Mr. Nelles.

An improvement by canalization, depending upon the natural conditions, can be effected: By the construction of canals with proper locks around the obstructed sections; by the construction of locks and dams, either fixed or movable, in the bed of the river, so disposed as to produce the desired depths; or by a combination of these methods. Considered simply as a means of improvement, canalization is a much more certain and satisfactory method than regulation for streams of comparatively small flow and protracted low-water periods.*

The method adopted for the improvement of the Warrior system appears to be admirably suited for the conditions and requirements, for, in addition to the reasons given for the adoption of this method of improvement, it can be shown, by the application of the usual hydraulic formulas, that a low-water flow of not less than 2 500 cu. ft. per second would be necessary in order to secure anything like the same results by regulation methods.

Fixed or Movable Dams.—The question of using fixed or movable dams no doubt received the consideration its importance demands, and the conclusion reached, to use fixed dams, was doubtless fully justified; still, it appears to the writer, from the information furnished by the paper, that this question is at least open to argument. With fixed dams, it is extremely doubtful, in a river characterized by rapid rises of such short duration, if there will be any open-river navigation at all; that is, it will not be practicable for boats to take advantage of the high-water periods, and increase correspondingly the size and draft of their tows, so that, as a result, the size of the locks and the depth on the miter-sills will be the limiting features of the commerce of the river. Consequently, if, as the author states, movable dams would add three months to the season of open-river navigation, it appears that the additional cost of construction and for maintenance would be fully justified, because the capacity of the streams would be increased sufficiently thereby to offset fully any reasonable difference in the original cost or for maintenance.

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* For a more complete discussion of the question of improving non-tidal rivers, and the methods recommended for effecting such improvement in special cases, the reader is referred to the following published reports by the writer:

	House Document.	Congress.	Session.
The Improvement of the French Broad River, Tenn.	No. 616.	56th.	1st.
The Improvement of the Mountain Section, Tennessee River	" 461.	56th.	1st.
The Improvement of the Little Tennessee River, Tenn.	" 66.	56th.	2d.
The Improvement of the Clinch River, Tenn.	" 75.	56th.	2d.
The Improvement of the Hiwasee River, Tenn.	" 77.	56th.	2d.
The Improvement of the Holston River, Tenn.	" 218.	56th.	2d.
The Improvement of the Middle Tennessee River	" 50.	57th.	1st.

Mr. Nelles. It is not thought by the writer that the leakage or loss of available water would be materially greater with movable dams than with the timber crib dams proposed. It has been shown by B. F. Thomas, M. Am. Soc. C. E.,* that it was possible at the needle dam in the Big Sandy River at Louisa, Ky., to reduce the leakage through the navigable pass, under a head of 12 ft., to less than 5½ cu. ft. per second, or less than 0.04 cu. ft. per second per linear foot of dam. In view of the comparatively small low-water flow in the Big Sandy River (50 cu. ft. per second) this loss would be of much greater relative importance in that case than a much greater loss in the case under consideration. The successful use of movable dams on the Kanawha and Big Sandy Rivers, and on the Meuse and other European streams, of far less low-water discharge than the Warrior, indicates clearly that leakage would not prove as serious a difficulty in the present case as anticipated by the author.

Stability of the Walls.—From an examination of the text and accompanying cuts (which, in the opinion of the writer, have been so reduced, condensed and abbreviated for publication that their usefulness has been materially impaired), it is not evident that the effect of submergence and upward pressure on the base has been considered in the design of the lock walls; doubtless, these features received consideration, but, to what extent, and how their effect has been provided for, is not made clear in the paper. Under ordinary circumstances, it is customary, in the design of retaining walls or dams, subject to water pressure, and founded on solid rock or other impervious strata, to neglect the effect of submergence and upward pressure on the base, on the supposition that by careful workmanship these effects can be reduced so that they need not be taken into consideration. Eminent authorities are arrayed on both sides of this question. In a paper entitled "High Walls or Dams to Resist the Pressure of Water,"† by the late James B. Francis, Past-President, Am. Soc. C. E., occurs the following:

"Most ledges of rock have seams in various directions which admit the passage of water. If such is the case with the rock on which the wall we are considering is built, we must assume that the water in the seams is in communication with the water in the reservoir, and that near (the back edge of the wall there) is an upward pressure on the masonry equal to the head. * * * The upward pressure may also extend over the whole base of the dam, naturally diminishing toward the toe, * * * where it would be zero. Supposing it to diminish uniformly from (the back to the toe), the average would be equal to that due to half the head. * * * The center of the upward pressure will be at * * * one-third of the width of the base from (the back edge). * * *

"It is often the case that in order to reach a suitable rock foundation a considerable part of the height of the wall is below the permanent

* Annual Report of Chief of Engineers, U. S. A., 1898, p. 2146.

† *Transactions*, Am. Soc. C. E., Vol. xix, p. 147.

level of the water in the ground. In this part of the wall the effective weight per cubic foot of the masonry is only its excess above the weight of a cubic foot of water." Mr. Nelles.

In a paper entitled "Notes on High Masonry Dams,"* by John D. Van Buren, M. Am. Soc. C. E., a strong argument is advanced in favor of designing dams to resist the probable upward pressure on the base.

In discussing these papers, Edward Wegmann, M. Am. Soc. C. E., denies the necessity for providing against the upward pressure, other than by careful workmanship, but admits that if the water should penetrate between the foundation and the base it would doubtless give rise to an upward pressure and diminish the stability of the wall.

Joseph P. Frizell, M. Am. Soc. C. E., says:

"Engineers of standing have affirmed that in computing the stability of cement masonry under water, when it is bedded on firm rock, the buoyancy of the mass should not be considered, the water being excluded from the bottom by the mortar. These experiments (by Mr. Francis) show that such exclusion is impossible."

E. Sherman Gould, M. Am. Soc. C. E., says:

"I think it may be doubted if the concluding paragraph of Mr. Francis' paper, regarding the loss of weight of foundations carried below the permanent level of water, is to be taken as an unquestioned fact. It would of course be safest, in such cases, to make calculations on both assumptions, and take the result which indicated the heaviest work."

E. A. Fuertes, M. Am. Soc. C. E., says:

"But it cannot be admitted as a certainty that 'an upward pressure may be transmitted through the mortar to the entire base' of the dam, for such a pressure could be transmitted only when the entire base is lifted from its support; and in this case no pressure could be exerted on the bed of the foundation. So long as the specific gravity of the material of the dam exceeds that of the water under the ordinary conditions of dams, direct contact and pressure must exist at many points of the base, however irregularly distributed or restricted they may be to small and unconnected areas. This fact points to the necessity of providing for increased surfaces, to diminish the probability of overloading the points of contact, thus increasing the probability of making them more numerous."

A paper,† descriptive of certain experiments by Messrs. Broenniman and Ross, shows clearly the existence of hydrostatic pressure in masonry of almost every description when subject to a head of water. The effect of this permeability on homogeneous structures and foundations is not set forth clearly in this paper or in the paper by Mr. Francis. It is generally conceded, however, that the stability of the wall is to some extent decreased.

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* *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 493.

† *Journal* of the Western Society of Engineers, Vol. ii, p. 449.

Mr. Nelles. In his "Practical Designing of Retaining Walls," William Cain, M. Am. Soc. C. E., writes as follows:

> "As water often saturates the filling, and perhaps gets under the wall, we must consider, in certain cases, water pressure in connection with the thrust of the backing. * * * If the wall is founded on a porous stratum, the weight of masonry is similarly reduced (when submerged) by 62.4 lbs. per cubic foot, or, say one-half, ordinarily; but if the foundation is rock or good clay 'there is no more reason why water should get under the wall than that it should creep through any stratum of well constructed masonry or puddle dam,' as Mr. Baker has observed."

From a review of the authorities quoted, and others, it is seen that, however much engineers may differ in regard to the effect of submergence and upward pressure on walls resting on an impermeable foundation, they are practically in accord as to the necessity for considering their effect when the foundation is permeable.

In the case under consideration, where the walls are all founded on a permeable stratum of sand or gravel, and are dependent entirely upon two rows of sheet-piling to cut off the water, there seems to be no room (whenever there is a head of water) to doubt the existence of an upward pressure on the base, and these designs have recognized the existence of such a pressure by providing openings through the wooden floor to relieve it.

Testing the stability of the river wall (Section F–F, Fig. 6), for a head of water of 17 ft. on the outside, when the lock chamber is empty, considering the upward water pressure on the base and the downward as well as the horizontal water pressure on the back, and assuming the wall to be 30 ft. high, 6 ft. wide on top and 12.25 ft. wide on the base, with sloping back, it is found that, for a weight of 140 lbs. per cubic foot for the masonry, the factor of safety against overturning is 2, instead of 6, as stated by the author, and that the center of pressure falls about 1.25 ft. beyond the outer limit of the middle third of the base, or at about the inner limit of the outer fifth of the base. According to Trautwine and others, these are safe conditions for well-constructed retaining walls, but, according to the generally accepted conditions of stability, *viz.*, that the center of pressure shall fall within the middle third of the base, and the factor of safety against overturning shall not be less than 3 for walls of this description, subject to water pressure and to sudden shocks and jars, it appears that a sufficient margin of safety has not been provided.

It is the writer's opinion that a much more trying condition of pressure against this wall will be found to exist during floods, when the lock is nearly full of water and the pressure is outward. The data are not given for an exact determination of this question, but, assuming the difference in level, at the time the walls are flooded, to be 2.5 ft., and that this difference increases proportionately up to

10 ft. as the river falls, and calculating the stability of the wall for the various changes in the pool levels, it is found that the most trying condition of pressure will exist when the water level in the lock is at Elevation 99 and in the river at Elevation 93.8. For this condition it is found that the factor of safety against overturning is only 1.64, and that the center of pressure is about 1.35 ft. beyond the outer limit of the middle third of the base. If the actual cross-section of the wall is used in calculating its stability, thus increasing the head of water by 1 ft. and the width of the base by 2 ft., it is found, for the conditions of pressure above stated, that in both cases the factors of stability will be less than shown by the preceding calculations. Mr. Nelles.

From these considerations it appears that this wall is safe, although the factors for the most trying conditions of pressure are not as great as the best practice demands for walls subject to shocks or jars from any cause, or where great interests would be jeopardized by a failure.

Testing the stability of the land wall, which is taken as 30 ft. high, 6 ft. wide on top and 16 ft. wide on the base, with five 2-ft. steps on the back (Section F-F, Fig. 6), for the case where the lock is empty and the head of water behind the wall is 17 ft., assuming, as is the general practice, that the wall will be called upon to resist the combined action of the earth and water pressures, estimated separately, and, in addition, an upward pressure on the base due to the head of 17 ft., and calculating the earth pressure by the Rankine formula, using a natural earth slope of 30° and a weight of 120 lbs. per cubic foot, it is found that the factor against overturning is 1.6, and that the center of pressure falls 2.3 ft. beyond the outer limit of the middle third of the base. For earth pressure alone the factor against overturning is 2.8, and the center of pressure falls close to the outer limit of the middle third of the base.

The writer has frequently had occasion to test the stability of gate abutment walls, and has found almost invariably that the dimensions of such walls as fixed by a consideration of the room required for operating machinery and for culverts, provided ample weight and stability to resist safely the external forces acting on them. In the present case, taking the lower river abutment, and assuming a length of 30 ft. to resist the thrust of the gate, for the condition of pressure for which the river wall was tested, *i. e.*, water in the lock chamber at Elevation 99, and outside the chamber at Elevation 93.8, it is found that the factor against overturning is 3 and against sliding is nearly 4.

Sill Failures.—One of the most fruitful sources of accident to locks is found in the lifting or failure of the miter-sills, due to insufficient weight or fastening. In this country, the most notable examples of accidents from this source that have come to the writer's notice are as follows:

At the Louisville and Portland Canal locks, in 1891, before the

Mr. Nelles. canal had been opened for traffic,* under a head of 9½ ft., the upper, middle and lower gate-sills failed in rapid succession by lifting. The sills were composed of large blocks of stone, 6 x 6 x 2 ft., laid in cement mortar on bed-rock, but not bolted. After the accident it was discovered that the bed-rock was full of seams, and had been shattered badly in preparing it for the foundation. The repairs were made quickly and successfully by bolting the sill stones to the bed-rock with 2-in. fox-wedged bolts of considerable length.

At the Des Moines Rapids Canal, in 1877, after having been in operation for one month, it was discovered that the coping courses of the lift walls and the lower sills of the middle and lower locks had lifted from 1 to 3 ins., under the pressure due to a head of 11 ft. When the locks were pumped out for repairs it was found that the primary cause of the failure was the imperfect leading of the holding-down bolts. The sills of all the locks on this canal were immediately strengthened and repaired by means of 1¾-in. fox-wedged bolts, extending well down into the bed-rock, and, while the bolts were still hot, the holes were poured full of melted sulphur. It is understood that these repairs were effective, and that no further trouble has been experienced from this cause.†

At the new 800-ft. lock at Sault Ste. Marie, in September, 1895, before the lock was opened to navigation, it was found that the coping of the upper guard-gate miter-sill rose 2½ ins. under the pressure due to a head of 14½ ft. of water. No reason is given for the failure, and the damage was immediately repaired by re-setting the stones and making them fast with one hundred 1½-in. fox-wedged bolts, set in rich mortar. The other sills at this lock were also re-bolted and made much more secure than was originally thought necessary.‡ Considerable difficulty was also experienced at this lock by reason of the lifting of the floor and culvert timbers when under pressure. This difficulty was remedied by the use of 1½-in. fox-wedged bolts of proper length. In connection with these repairs, a great many tests and experiments were made to determine the holding power of anchor bolts set in cement mortar.§ The results of these tests may be stated briefly as follows: The adhesion of 1 to 1 Portland cement mortar to hammered bar-iron varied from 42 to 152 lbs. per square inch, in six months. The adhesion of 1 to 2 Portland cement mortar to 1-in. round iron rods, imbedded from 1 to 9 ins., varied from 230 to 350 lbs. per square inch, in one month. Tests made with limestone screenings instead of sand in the mortar gave results nearly double those above stated. The tests made seem to indicate that the adhesion per square inch is independent of the area in contact, whether due to the depth imbedded or to the size of the rods.

* Annual Report of Chief of Engineers, U. S. A., 1872, p. 460.
† Annual Report of Chief of Engineers, U. S. A., 1877, p. 736.
‡ Annual Report of Chief of Engineers, U. S. A., 1896, p. 2787.
§ Annual Report of Chief of Engineers, U. S. A., 1895, p. 2917.

At the Muscle Shoals Canal, in Northern Alabama, the same trouble was found at all the locks where the lift exceeded 8 ft. The sill stones are generally 6 x 6 ft., and 2.5 ft. thick, set in mortar on bed-rock and partially bolted. No serious accidents have ever resulted at these locks from this cause, and the repairs have been effected by re-bolting the disturbed sill stones and grouting with rich mortar. Mr. Nelles.

In view of these somewhat general defects in lock design or construction, the writer is of the opinion that some special provision, not indicated on the plans, will be necessary to assure the safety of the gate and coffer-dam sills in the design now under discussion.

Gates, Valves and Culverts.—The adoption of the methods advanced by the United States Deep Waterways Commission in the design of these gates should make them an example of the most advanced ideas in structures of this kind, and it is to be regretted that the author has not gone deeper into the details of the design, and given the dimensions, conditions of pressure and size of parts on the drawings. The method of operation and the details of the gates have been worked out in a highly ingenious manner.

The exact design of the gates for locks of this description, where the conditions of pressure are constantly changing, is much more difficult and complicated than the design of gates where the pressure conditions can be considered constant, as for instance in the case of the Deep Waterways Commission locks. In cases like the present one, where the pool levels are constantly changing with the stage of the river, each part should be designed especially for the most trying condition of pressure that can come on it, and it is not unusual to find that this condition is different for each member in the gate; consequently, it is not possible to design such a gate with reference to its rigidity, as advocated by Captain Hodges in his work on "Mitering Lock Gates," but rather with reference to a given unit stress under the maximum load.

The variation between the actual and theoretical stress in lock gates, as determined by the usual methods of calculation, and the effect of the vertical members in distributing the stress through the gate, is well illustrated by certain observations made in 1898 by Sydney B. Williamson, M. Am. Soc. C. E., at Lock 6 of the Muscle Shoals Canal. These observations consisted in measuring the actual deflection of the horizontal girders in the lower gate at Lock 6, with the lock full and the lower pool empty. This gate is made up of 15-in., 53-lb., rolled I-beams, 34.6 ft. long, for horizontal girders, spaced as shown in Table No. 8. The gate is sheathed on the up-stream side with ½-in. plates, which extend in unbroken sheets from top to bottom of the gate, with ½ x 12-in. cover plates at all joints, and is stiffened by means of four diagonal 9-in., I-beams, weighing 25 lbs. per foot, arranged in the form of an inverted W, riveted on the back of the

Mr. Nelles. gate. The quoin and miter-posts are fastened to the sheathing and up-stream flange of the horizontal girders in such a manner that they all act together.

TABLE No. 8.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Feet.	Pounds.	Pounds.	Inches.	Inches.	Pounds.	Pounds.
0.21	30	808	0.07	0.56	6 315	234
4.22	880	23 725	2.10	0.94	10 60C	393
6.86	1 210	32 622	2.892	1.125	12 686	470
9.85	1 670	45 024	3.992	1.125	12 686	470
12.29	1 825	49 203	4 363	1.06	11 953	443
14.60	2 060	55 539	4.925	0.75	8 457	313
16.81	2 220	59 853	5.307	0.63	7 104	263
17.81	Top of	sill.				
18.82	1 430	38 554	Bottom of	gate.		

Table No. 8 shows: (1) the head of water on the horizontal girders, *i. e.*, the depth of the girders below the level of the upper pool; (2) the load per linear foot on each horizontal girder, calculated on the assumption that the gate has no vertical rigidity, and that the pressure on the sheathing due to the head is transmitted to the nearest horizontal girder; (3) the corresponding maximum unit stress in the horizontal girders, on the supposition that they carry all the pressure, and neglecting the effect of the thrust in the horizontal girders; (4) the calculated maximum deflection corresponding to the maximum unit stress given in Column 3; (5) the actual measured deflection of the horizontal girders; (6) the calculated maximum unit stress corresponding to the measured deflection, neglecting the effect of the trust in the horizontals, and assuming that the girders receive no assistance from the sheathing; (7) the calculated load per linear foot corresponding to the deflection and unit stress given in Columns 5 and 6.

Under the usual assumptions and methods of calculating the stress in gates of this kind, it appears from Column 3 that this gate is absolutely unsafe, and is strained beyond the elastic limit, yet the measured deflections and the actual condition of the gate show that such is not the case.

By assuming the sheathing to act with the compression flange of the horizontal girders, and the compression due to thrust in the girders to offset the excessive tension in the other flange, the calculated stress can be reduced to safe limits; but it is not possible, by any reasonable assumption, to reduce the calculated stress to the figures corresponding to the measured deflection.

Power to Maneuver Lock Gates.—The resistance to be overcome in maneuvering lock gates is made up of the frictional resistance of the bearings, the wind pressure against the exposed surface of the gate,

and the resistance due to the dynamic pressure of the water. All of these resistances can be calculated with reasonable certainty for given conditions. Under conditions requiring the operations of the lock gates while the wind is blowing 15 miles or more per hour, it is usually found that this is a source of far the greatest resistance to be overcome, and that machinery designed to overcome the resistance due to wind pressure will be more than sufficient under all ordinary circumstances. Mr. Nelles.

In the study of the power plant for the lift lock of the Colbert Shoals Canal, the writer found the probable maximum force required to maneuver the gates to be made up as follows:

Dynamic resistance........................	15 per cent.
Wind resistance (30 miles per hour)..........	75 "
Frictional resistance.......................	10 "

The actual power required depends upon the method of application, the character of the bearings and contact surfaces, and the time allowed for the operation, and may vary from that exerted by one man through simple and inexpensive appliances to 40 and 50 H.-P. exerted through expensive and complicated steam, hydraulic or electric motors.

At the Canadian "Soo" Lock, one 25-H.-P. electric motor is provided for each 37 x 40-ft. gate leaf, and one similar motor for each pair of 8 x 8-ft. butterfly valves.

At the American "Soo" Lock, a total of 150 H.-P. is provided to operate the hydraulic motors for four 40 x 50-ft. gate leaves and four 8 x 10-ft. culvert valves.

At the Cascade Locks, Ore., where the machinery is operated by direct water pressure, it was found that an actual expenditure of 10, H.-P. was necessary to open one 40 x 50-ft. leaf weighing 125 tons. and under ordinary circumstances; and the opening of the 10 x 10-ft. culvert valves required an expenditure of 18 H.-P.

The gates at Lock "A" of the Muscle Shoals Canal are operated by special hydraulic engines of unusual form and construction. In these engines the water pressure is applied against the sides of a small vane or fin, 9 x 18 ins., fastened to the quoin post in the extension of the axis of the gates, and working in a fan-shaped box arranged so as to allow the application of the pressure on either side of the vane.

The lower gates are 18 x 37 ft., and, when the wind is not high, can be opened and closed in 1½ minutes under 70 lbs. pump pressure. Under less favorable circumstances a pressure of 150 lbs. is often required to start the gates. This appliance has not proved perfectly satisfactory on account of the frequent breakage of the operating vane or fin.*

* Illustrations, showing the details of these engines, may be found in the Annual Report of the Chief of Engineers, U. S. A., 1890, sheet 111, p. 2126.

Mr. Nelles. *Culverts.*—The considerations bearing on the designs of culverts for filling and emptying locks are quite fully set forth in the recent report of the United States Deep Waterways Commission, and in that report reference is made to the wide variation in the coefficient of discharge through lock culverts, due to the shape and design of culverts.

Table No. 9 shows some rather crude observations, made at the Muscle Shoals Canal, which are of some interest and value in this connection. It gives the coefficient of discharge, for both filling and emptying locks of the usual workmanship, through culverts designed without special reference to ease of flow.

TABLE No. 9.

Lock number.	Horizontal area of locks.	Lift of locks.	Time to fill locks.	Time to empty locks.	Area of culverts.	Coefficient while filling locks.	Coefficient while emptying locks.
	Square feet.	Feet.	Seconds.		Square feet.		
1.......	18 216	3.35	300	300	44.56	62.2	62.2
2.......	18 216	7.2	420	480	38.72	74.9	65.5
3.......	17 672	11.4	540	720	40	68.9	51.6
4.......	17 676	10.35	600	540	40	58.4	64.9
5.......	17 974	11.8	600	540	44.56	57.6	64
6.......	17 960	12	660	480	44.56	52.8	72.5
7.......	17 980	11.6	540	450	44.56	63.5	76.2
8.......	17 980	9.8	450	480	44.56	70	65.6
9.......	18 113	6.3	300	360	44.56	86.8	70.7

The time given in the table begins with the opening of the culvert valves and ends when the gates could be freely opened.

All of these locks, except Nos. 2 and 3, have two short filling and emptying culverts, about 4 x 6 ft., built in the gate abutments around the gates. Locks 2 and 3 have single 6 x 8-ft. culverts. The valves in the culverts reduce the area to that given in the table.

Size and Capacity of the Locks.—At first sight it seems that the locks are of insufficient size (52 x 282 ft.), to accommodate a large traffic, but, by an analysis of the conditions and methods, it can be shown that, under favorable circumstances, the system will have a sufficient capacity to handle between 8 000 000 and 10 000 000 tons of coal down stream per annum, in addition to carrying the empty barges back and taking care of the probable inland commerce. This is more than the present coal traffic on the Mississippi River, and is probably more than the Warrior system will be called on to carry for a great many years, and possibly more than the North Alabama coal fields will be able to produce.

In conclusion, the writer wishes to disclaim all intention to criticize the improvement or designs described by the author, but offers these remarks simply as a discussion of the questions opened up by the paper. Mr. Nelles.

EDWARD P. NORTH, M. Am. Soc. C. E.—Neglecting the technical portion of Mr. McCalla's valuable and interesting paper, and turning attention to the economic questions developed, occasion is taken to reiterate the opinion that unless a waterway is improved in such a manner as to pass boats of a burden at least equal to that of freight trains on parallel railroads, such an improvement will fail of its full economic service. In other words, the improvement of a watercourse may be of doubtful value unless it is on so ample a scale as to develop new industries and sources of freight by offering such low rates for transportation that commodities otherwise unsalable can be produced and brought to market with profit. Mr. North.

Through such a service consumers receive a double benefit in the reduction in price, due both to increased supply and the reduction in cost of transportation; much labor and capital is profitably utilized in production that would otherwise be idle, and railroads find their traffic and profits augmented by increased receipts from passengers and package freights. A smaller waterway, however, may be valuable in reducing freight rates on competing railroads, as set forth in reference to the Erie Canal by the late Albert Fink, Past-President, Am. Soc. C. E. But any such reduction, advantageous as it may be for producers and consumers, may be made to the loss of net income by the railroad, unless it is so radical as to develop new industries.

The history of the traffic through and past the "Soo" may be as instructive as to the value of large channels for transportation as any known. The last report of Colonel G. J. Lydecker, on "Lake Commerce Passing through Canals at Sault Ste. Marie, Michigan and Ontario, for 1901," gives data from which Table No. 10 has been made.

TABLE No. 10.—"Soo" TRAFFIC.

Year.	Tons of freight.	Aggregate value.	Value per ton.
1851	12 600	1 675 000	$133.00
1861	88 000	6 000 000	68.00
1871	585 000	13 000 000	22.00
1881	1 568 000	30 000 000	19.00
1891	8 889 000	128 000 000	14.50
1901	28 403 065	289 916 865	10.21

That is, in 1851, without a canal, the produce of that country could not be brought to market unless its average value was $133. Fifty years later, with a canal, an average value of $10.21 brought more than 2 200 times as much freight to market.

Mr. North. It will be remembered that the canal was opened in 1855 with double-lift locks, having $11\frac{1}{2}$ ft. on their miter-sills, built under charter from the State of Michigan. In 1881 a single-lift lock, with 17 ft. on its miter-sills, was built by the General Government. In 1895 the Canadian lock, with 20 ft. on its miter-sills, was opened, and in 1896 another lock by the United States Government, known as the Poe lock, was opened to traffic, with 21 ft. on its miter-sills.

It will be noticed immediately that up to 1871 the canal was doing an immense and rapidly increasing service to the country by increasing the commodities brought to market and decreasing the cost at which they could be offered to consumers. But between 1871 and 1881 there was a marked slackening in the public value of the canal, both as to quantity carried and reduction in value of freight, followed, after the last-mentioned date, by an increasing value in its services. During the decade ending with 1881, or, more exactly, during its last half, the limiting depth of the miter-sills in the "State locks" prevented profitable competition with the railroads, and Lake commerce was diminishing. But, opening the 17-ft. lock and the concurrent deepening of the Lake channels has resulted in the growth indicated.

This relation between traffic and the capacity of its channels is ably discussed in the "Report of the Committee on Canals of New York State, 1899," made to Governor, now President, Roosevelt. Commencing on page 196, it says:

"In 1875 the total tonnage on the lakes was nearly 600 000 tons. * * * The traffic, as estimated by entrances and clearances at American ports, was for that year about 15 000 000 tons.

"This development had, however, been secured without any material increase in the size of the vessels in use—a limit being placed on size by the depth of water in the harbors and in the channels connecting Lake Erie and Lake Superior with Lake Huron-Michigan. By 1875 the advance in railroad construction and management had reached a point which enabled the railroads to compete with the lake vessels; and when to this was added the railroad rate wars of the next few years there was not only a cessation of increase in lake commerce, but a positive decline both in equipment and traffic. New construction of vessels, which had reached as high as 73 000 tons in 1874, was only 7 000 tons in 1877, and averaged only 13 000 tons a year for the five years ending in 1880. In the latter year all the vessels on the lakes aggregated only 560 000 tons, 30 000 less than five years before.

"The beginning of the new decade marked a significant revival in lake commerce, which cannot be disconnected from the improvements in lake harbors and channels undertaken by the national government. The most important work brought to completion at this time was the opening of the ship canal with an 18-ft. draft, which took the place of the 10-ft. canal and locks maintained by the State of Michigan at Sault Ste. Marie."

Placing the depth at 10 ft. is an unfortunate misprint. On page 301 of the Report of the Chief of Engineers for 1881, we read:

"A depth of about 11 ft. 6 ins. could be carried through the channel between Lakes Superior and Huron when the Government

began its improvement in 1870 * * * to construct a new lock with a single lift of 18 ft. and a capacity of 515 x 80 ft., for a draught of 17 ft." Mr. North.

And it adds that, up to that time, the expenditures had only resulted in greater speed. In the same report for 1882, page 2358, Major Weitzel says that the conditions of the land grant, by the aid of which the State of Michigan built the canal, called for a depth of water of 12 ft., and he states that the canal had a depth of 12 ft. at mean stage.

Incidentally, Major Weitzel estimates a fair freight rate per ton-mile on iron ore through the State Canal as 2.8 mills. The average freight rate on all commodities returned for 1887 was 2.3 mills, and, with the facilities offered by enlarged channels, the average freight rates have been less than 1 mill, while the freight on iron ore for 1901 was 0.78 mill per ton-mile.

Continuing, the report, after calling attention to the increase, both of tonnage and in the size of vessels built, on the opening of the deeper lock, says:

"The increase of shipping was, of course, but the accompaniment of the increase of traffic. In 1885 the record of entrances and clearances at the United States ports on the lakes showed a traffic of 19 200 000 tons; by 1890 this had almost doubled, reaching the figure 37 500 000 tons, about 9 000 000 tons of which passed through the St. Mary's Falls canal. By 1896 the total traffic was 52 000 000 tons, and in 1898 62 500 000 tons, 18 000 000 tons passing through the St. Mary's Falls canal."

This is followed by an estimate of 60 000 000 tons as the total freight movement on the Lakes for 1898. Most of the figures given above are reiterated in diagrams and tables incorporated in the text of the report.

Increasing a decaying traffic of 15 000 000 tons in 1875 to 62 500 000 in 1898 is, as set forth in the report quoted, so plainly the result of increasing the maximum possible draft of vessels from 11½ to 17 and 20 ft. as to cause doubts if those calling for a 10-ft. navigation through New York State as improving its water routes "to the utmost limit of which it is capable," have ever seen the report of the Committee on Canals, or know that the Canadian canals, with 14 ft. on their miter-sills, have been unable to effect serviceable economies in transportation.

With the increase of freightage through the "Soo," the development of the country directly tributary to and feeding this route has been marked. Taking the "Upper Peninsula" of Michigan, Wisconsin, Minnesota and the Dakotas, the following figures are from census returns for the years named:

	1850.	1880.	1900.
Population............	317 013	2 315 477	4 802 585
Value of farms........	26 870 511	575 109 958	1 783 498 355

Mr. North. It cannot be claimed that all of this increase in population and farm values is due to the cheap transportation offered by the Lake route, though it will be readily admitted that it has been an important factor in the development of those States.

The development of our iron and steel industry, however, is universally admitted to be dependent on this water route. In 1855, the year the canal was opened, the Hon. A. S. Hewitt made a generally accepted estimate that the world's production of pig iron was 7 000 000 tons. Of this amount the United States contributed 700 159 tons, or 10 per cent. In that year 1 447 tons of iron ore were carried through the Canal. In 1880, 677 073 tons of iron ore passed through the lock at the "Soo"; the total production of pig iron is estimated at 17 950 000, and the United States contributed 3 835 191 tons, or 21.3% of the total. For 1900 Mr. Swank estimates the total production at 40 400 000 tons, of which the United States made 13 789 242 tons, or 34.1%, and that season 16 443 568 tons of ore went down the Lakes through the locks at the "Soo." And it seems reasonable to predict that if a convenient waterway between Duluth and New York Harbor, with 21 ft. depth, could be secured, as asked by the various Deep Waterways Conventions, within ten years the United States would be making fully 50% of the world's output of pig iron.

The service done to this country, and the world in general, by the decreased cost of iron due to our large production, which is made possible by the commodious and cheap transportation through the enlarged waterways of the Lakes, does not receive the full recognition that is its due. In 1872, when for the three preceding years our make had not averaged 1 700 000 tons, the price of pig iron was run up by unfeeling monopolists, on an increased demand, to $53.88 per ton for the month of September, averaging $48.88 for the year. Again, during the boom of 1880, when we made only 3 835 191 tons of pig iron, the demand for steel rails exceeded the supply, with an official price of $85 in February and an average of $67.50 for the year. This year, with our possible, if not probable, make of 18 000 000 tons, and the largest consumption of iron ever known, the published price of steel rails is $28, though more is paid for prompt delivery, and the high prices of the past are improbable unless our waterways are allowed to decay. There seems to be no ground for doubting that the improvement of the Lake channels has been of great service to the people of this United States in particular, and the world in general, by developing industries and creating new sources of supply. The only reason that the improvement should not be continued to greater depth seems to be that the value of capital invested in the boats now in use would be impaired by the lower freight rates at which larger boats could be profitably operated.

The value of improvements on the Mississippi River are not as unmistakable as those on the Lakes. This river is badly handicapped

by the fact that it has the least water in its channels during the season of the greatest freight offerings. On the other hand, it is nearly 80% longer than the aggregate of the routes from Chicago and Duluth to Buffalo and has 15 000 miles of navigable tributaries, most of which are shallow streams. But, according to a paper* by J. A. Ockerson, M. Am. Soc. C. E., 8 ft. cannot be depended on, and there is a recorded depth of 3 ft. between St. Louis and Cairo. In high water, however, cargoes of 40 000 to 50 000 tons of coal are taken in a single tow from Louisville to New Orleans, "a distance of 1 400 miles, at a cost of about 10 cents per ton." Mr. North.

While Mr. Ockerson does not deal with all the traffic of the river, he gives a diagram showing the main features of the trade between St. Louis and New Orleans for the thirty years ending with 1900, and a very valuable table, commencing with 1865. This table shows that for the three years ending with 1867 the river freight on a bushel of wheat from St. Louis to New Orleans averaged 29.4 cents; and for the three years ending with 1900 the average was 4.42 cents. By rail, from St. Louis to New York, the average freight charge for the first mentioned period was 65.4, and for the last, 12.7 cents. By river the fall has been 85%, and by rail it has been 80.5 per cent. The percentage of river traffic to the whole grain traffic has varied from 0.31% in 1870, the first year in which it is given, to a maximum of 37.9% in 1881, and for 1900 it was 6.4 per cent. The percentages of wheat carried by river have been erratic. The decline in freight rates has been fairly uniform for both routes.

These reductions in freight rates, of course, have added materially to the wealth of the Mississippi Valley, and cheapened the merchandise carried to consumers. Some measure of this service to the Mississippi Valley may be gathered from the history of the Illinois Central Railroad, which was consolidated with the Chicago, St. Louis and New Orleans Railroad in 1882, making a through line from Chicago to New Orleans. In 1883 the freight traffic of the road was 604 632 667 ton-miles, carried at 1.43 cents. Since that date the Ohio has been bridged at Cairo, with many extensions and improvements of track, etc., showing well-founded commercial confidence in the future of the road and the country it serves. For 1900 the freight traffic of the road was 3 425 794 698 ton-miles, carried for 0.65 cent. Under a decrease of 54.5% in earnings per unit, the freight earnings increased from \$3 463 648 to \$22 280 420.

Either the Mississippi River, the rail route between St. Louis and New York, or the Illinois Central Railroad, or all of them, have evidently been of great value to production in that portion of the country they serve, and if any damage has been done to either interest by the lessened cost of transportation, the greater burden seems to have fallen on the River interest, which still seems strong enough to

* Read before the Engineers' Club of St. Louis, September 18th, 1901.

Mr. North. prevent any increase in freight rates, and possibly strong enough to force a continued decline.

There are two striking instances of insufficiently improved waterways, *viz.*, the Erie Canal and the Tennessee River. When it was decided to build the Erie Canal, New York was a one-horse town, inferior in prestige and importance to either Charlestown, Philadelphia or Boston. Wise men at that time thought New London would be the principal seaport of the Union. From the opening of the Erie Canal, in 1825, until about 1880, its comparatively low freight rates and great volume of traffic made it an important factor in additions to our wealth, and caused all the railroads of the country to center at New York Harbor. But since 1862 its channel has not been enlarged. For twenty years competing freight trains have been carrying more than fully loaded boats. Though there is yet a slight reduction in grain rates when the canal opens, it is powerless to prevent discriminations against the Port of New York and the inhabitants of that city, and the interests of the State and country are so overshadowed by the influence of those who keep toll houses at its ends that no proposition to increase its navigable depth to more than 10 ft. seems to be entertained.

In the Tennessee River, the great obstacle to navigation is at and about the Muscle Shoals, where the river falls more than 100 ft. Some 16 miles of canal, with a steel trunk 900 ft. long and having a cross-section of 60 x 5 ft., and with locks 300 x 60 x 5 ft., were opened on November 10th, 1891, at a cost of slightly more than $3 000 000. In 1900 traffic had developed to such an extent that 14 881 tons of freight passed through the canal, at a cost of $65 546 for attendance and maintenance. This is $4.40 per ton, and, at the average railroad rate for that year, would have carried the freight nearly 600 miles. The unremunerative character of the improvement is due to the fact that no boat which can pass through the locks can float machinery enough to make it effective as either a freight carrier or a towboat. The Tennessee is nearly as long as the Ohio, and has the potentiality of nearly as great wealth.

Neither of these instances presents any argument against improving waterways. The Erie Canal, which once controlled the route of freight and the location of wealth by presenting the most ample and convenient channel for transportation, has fallen to its present low estate greatly through political and personal aspirations. The money expended on the Tennessee has been wasted, to a great extent, and the development of wealth in its valley seriously retarded by men of undoubted honesty, but possessed of inadequate knowledge of the transportation problem.

Mr. McCalla does not mention the tonnage of the boats that are expected to pass the locks on the improvement he describes, but it is possible that they will accommodate barges of from 1 800 to 2 000 tons, or about one-half of the probable competing train-load. The small

draft will prohibit distribution along the shores of the Gulf, without transferring cargoes, and, as the locks will only admit one boat at a time, fleet towing will be attended with considerable delay and expense, except on the lower fourth of the improvement. It would cheapen transportation materially if the three lower locks were lengthened so that tows could be made up for Mobile at Demopolis. As at present, it is questionable if the improvement is of sufficient capacity to develop new industries or serve the country by reductions in freight rates material enough to cause noticeable increase of wealth in the valley of the Warrior and Tombigbee Rivers. Mr. North.

Improvements in the mechanism and methods of railroad transportation have had, and will probably continue to have, so marked an influence on transportation by water that reference is made to the beneficence of our railroad development, a great deal of which has been forced on their managers by the competition of our free waterways, and most of which would have been impossible without the increased production and consumption due to the low transportation charges found profitable by both water routes and railroads.

In 1882 the editor of "Poor's Manual" succeeded for the first time in obtaining the ton-mileage and rates on all of our railroads. The figures for the two extreme dates available are:

	Ton-mileage, United States Railroads.	Rate per ton-mile.
1882	39 302 209 249	1.236 cents.
1900	141 162 109 413	0.746 cent.

This is an increase of 259% in service, with a decrease of 39.65% in compensation per unit. This is not as good a showing as is made by the Lake traffic, but, considering the volume moved, it is probably more efficient in wealth production.

A comparison of our results with those attained in the United Kingdom of Great Britain and Ireland is rendered difficult by the fact that the English do not return ton-mileage, but, as there is neither a free internal water route, a free harbor, nor any long canal of even the capacity of the Erie Canal in that country, a comparison is made in Table No. 11 on the basis of tons shipped.

TABLE No. 11.—Freight Shipments.

Railways of the United Kingdom.

	Freight receipts.	Tons shipped.	Receipts per ton.
1882	£37 704 315	256 216 833	35.32 pence.
1900	53 470 564	424 929 513	31.20 "

Railroads of the United States.

	Freight receipts.	Tons shipped.	Receipts per ton.
1882	$485 778 331	360 490 375	$1.348
1900	1 052 835 811	1 071 431 919	0.983

Mr. North.

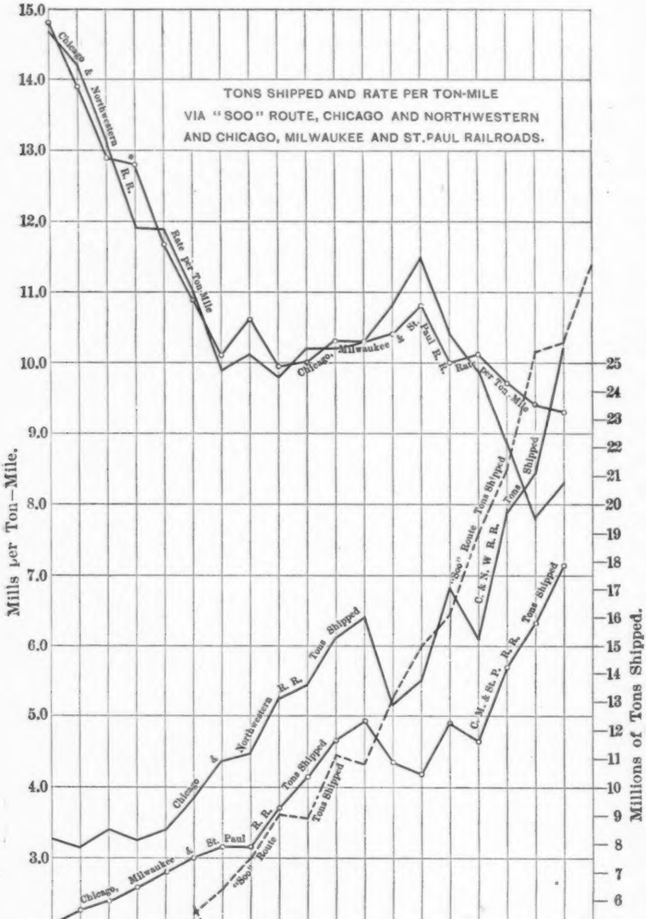

FIG. 32.

Mr. North.

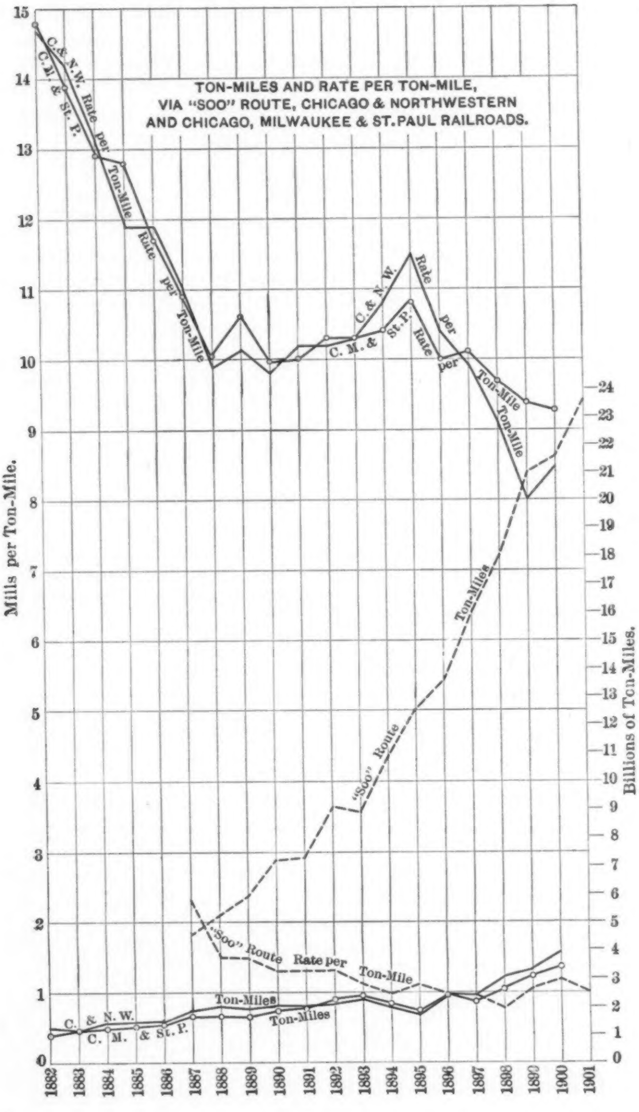

FIG. 33.

Mr. North. It is seen that, while the British managers have increased the tons shipped by approximately 66% and decreased the compensation by 12%, our managers have increased the tons shipped by 170% and decreased the charges per ton shipped by 27 per cent. One result of the more liberal policy in this country is shown in the following:

TONS SHIPPED PER CAPITA.

	Year.	Tons.	Year.	Tons.
United Kingdom................	1882	7.28	1900	10.25
United States..................	1882	6.48	1900	14.05

That is to say, in 1882 the inhabitants of the United Kingdom shipped 6.4% more tons per capita than we, and in 1900 we shipped 37% more than they. Concurrently, the recognized value of our railroad securities as investments is changing places with those of the British. The comparison is not entirely satisfactory, as it is generally understood that for more than fifty years there has been no substantial reduction in British freight rates; J. S. Jeans saying, in his "Railway Problems," that any apparent reduction in English rates is due to shortened haul; but in this country the average haul has been increased by about 20 per cent.

Speaking of our reductions in freight rates, the learned author says:

"This colossal concession to the trade and industry of America has both immediate and far reaching consequences. Its more immediate effects have been to enable the remotest cattle-breeder and wheat-grower in the United States to obtain access to other markets than his own, and thereby to enter into the world's competition for the supply of the world's markets. Its ultimate results are to be witnessed in the extraordinary cheapness of the bread-stuffs furnished to England by the United States, and, as a necessary consequence thereof, by other countries; in the singularly severe and protracted depression of British agriculture; and in the complete discomfiture of many interests that were fairly strong and capable of holding their own until this fiscal monster came to the front."

There was for some years a remarkable parallelism between the capital put into American railroads and the decrease in the value of English farm lands. How far the early adoption of our traffic and fiscal policies by the British would have averted "the complete discomfiture of many interests that were fairly strong, etc.," until confronted by a superior intelligence, will never be known, but the complete discomfiture of large interests, through a narrow minded and obstructive grasping for large gains, does not, on the whole, make for the general welfare, however just it may be for the obstructionists.

The effect of a capacious waterway on near-by railroads may be seen by comparing the route through the "Soo" with the Chicago and Northwestern, and the Chicago, Milwaukee and St. Paul Railroads.

Mr. North.

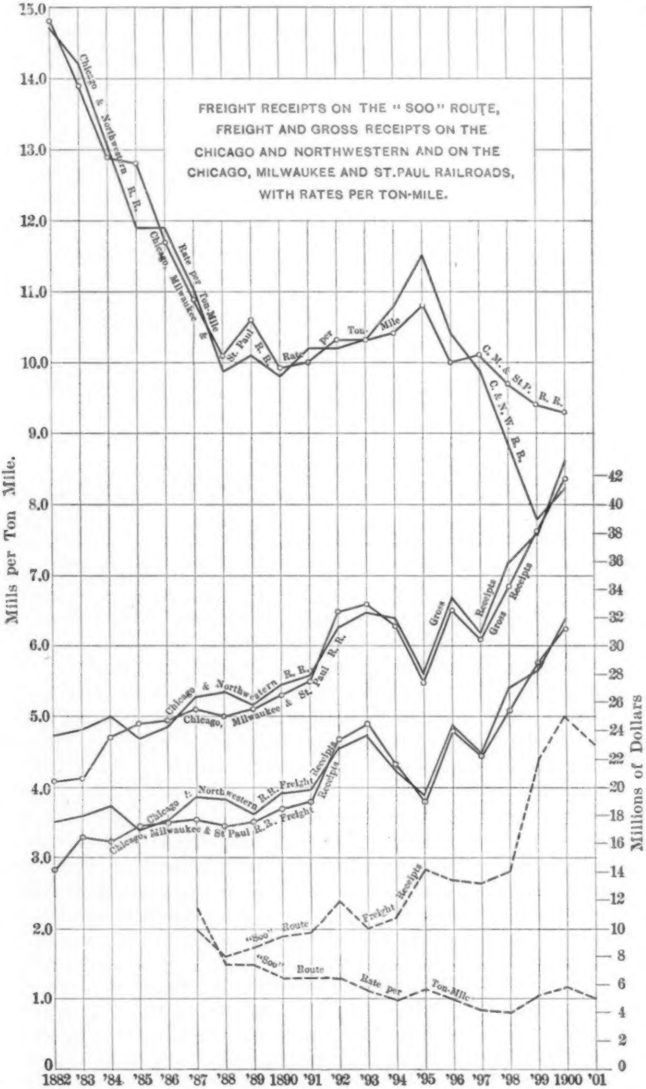

FIG. 34.

Mr. North. In 1901 the tonnage through the "Soo" was eighteen times that of 1881, and the average value per ton was between 53 and 54% of the value in 1881. No definite return of freights paid is known until 1887; then the average freight rate was 2.13 mills per ton-mile, and in 1901 the rate was 0.99 mill. The two railroads named, in a part of their lengths, compete with the "Soo" route, but through a large area they feed it and are fed by it. Commencing with 1882, the services they performed and the rates charged are given in Table No. 12.

TABLE No. 12.

For 1882.

C., M. & St. P. R. R.	945 250 159	ton-miles at	1.48 cents.
C. & N. W. R. R.	1 192 188 039	" "	1.47 "
	2 137 438 198	" "	1.475 "

For 1900.

C., M. & St. P. R. R.	3 357 456 584	ton-miles at	0.93 cent.
C. & N. W. R. R.	3 849 367 760	" "	0.83 "
	7 206 824 344	" "	0.873 "

Here the decrease in freight rates, though noticeable, is not as great as on the Lakes, nor is the increase in traffic as large. The increase in ton-mileage has not been quite up to the ratio of the whole country, and the rate charged has fallen a little faster than that ratio.

The effect of an average decline of 40%, in the charges made to the inhabitants of their territory for services rendered, on the value of the two roads, as shown by the market prices of their stocks on July 1st of the two years is shown in Table No 13.

It does not seem reasonable to claim the cheap transportation offered by the improved Lake route as the sole factor in the increased population and farm values in the States tributary to that route, nor can it be contended that the doubling of the value of these two great railroads would have been possible without the contributions to the wealth of the country made by the low freight rates on the Lakes. Both channels of distribution seem to have been mutually helpful.

The relations between the freight traffic, the receipts per unit, and the aggregate receipts, on the two roads mentioned and through the "Soo" are shown on the diagrams, Figs. 32, 33 and 34.

Attention should be called to the fact that, while the average haul on the railroads is about 160 miles, by the "Soo" it is more than 800 miles.

TABLE No. 13.—VALUE OF STOCKS ON THE C., M. & ST. P. AND THE C. & N. W. RAILROADS ON JULY 1ST, 1882 AND 1900. Mr. North.

JULY 1ST, 1882.

C., M. & St. P. R. R.	$27 904 261 Common, at 1.12	$31 252 772	
	16 447 483 Preferred, at 1.27⅝	20 989 702	$52 242 474
C. & N. W. R. R.	15 095 924 Common, at 1.31¼	$19 813 400	
	22 153 119 Preferred, at 1.46	32 343 552	52 156 952
	816 008 Shares, at 127.93	....	$104 399 426

JULY 1ST, 1900.

C., M. & St. P. R. R.	$47 146 600 Common, at 1.11	$52 332 726	
	40 454 900 Preferred, at 1.71¼	69 279 016	$121 611 742
C. & N. W. R. R.	39 114 678 Common, at 1.59⅛	$62 241 231	
	22 395 160 Preferred, at 1.96	42 894 514	105 135 745
	1 491 109 Shares, at 152.06	....	$226 747 487

As long as all charges for transportation must be taken in some ratio from the resources of both producer and consumer, and high freight rates are as inimical to the prosperity of a country as low rates of wages, the claim made by many railroad managers that reductions in freight charges are a gift to the public will be true, but it seems, in the instances cited, to be fully as much a gift to the railroads as to the community. On the other hand, the assertion sometimes made, that a reduction of rates will take a specified sum out of the pockets of the railroad making such reduction, seems as fallacious as figuring our ton-mileage at what Mullhall said was the English rate, 2.8 cents per ton-mile, and claiming the $4 000 000 000 difference between rates as a loss to American railroads.

S. WHINERY, M. Am. Soc. C. E.—There are one or two points to Mr. Whinery.
which the speaker wishes to call attention. In all these projects for improving waterways for navigation, the question of the quantity of business to be accommodated must be carefully considered. It is not sufficient to assert that the territory penetrated is capable of affording a large business by the development of its natural resources; there must be a demand for these products along the route or at its terminals. The great deposits of coal that are tapped by the headwaters of the Warrior and Tombigbee Rivers will undoubtedly all be mined in the future and will find a market. It is not at all clear to the speaker that this market will be found at the City of Mobile, or at any other points that can be served by river transportation better than by rail transportation.

Mr. Whinery. According to the census of 1900, Mobile had a population of a little less than 38 500. In 1890 the population was slightly greater than 31 000. There is no reason to believe that, in the future, it will become a very large city, and still less reason to expect that it will become a great manufacturing center. The local consumption of coal, therefore, is quite certain to be comparatively small.

To what extent coal may be exported from this country is yet problematic, and even if the export business should become very large it is not at all certain that Mobile would secure a very large share of that business. The city is at the head of Mobile Bay, some thirty miles from the open Gulf, and it is reached through a long stretch of artificial channel which will, doubtless, require the expenditure of large sums of money for maintenance, and which, at best, is somewhat difficult of navigation.

The construction and maintenance at Mobile of a harbor of sufficient capacity to accommodate a very large export business will involve a further very large expense.

The distance to European ports is notably greater than from Norfolk and other points on the Atlantic seaboard where coal is available for export. On the other hand, Mobile is much nearer to Mexican, West Indian and South American ports depending for their coal supply largely upon importation.

There are no large cities in the region that would probably be supplied with coal through Mobile, as New Orleans will probably continue to get the bulk of its supply by the Ohio and the Mississippi Rivers, and Galveston could probably be supplied more economically by rail. Therefore, there is no assurance of a market at Mobile that would justify the very large expenditures of money required to make a satisfactory water route between the Warrior coal field and that city. Nor is it likely that any great points of coal consumption will be developed in the intermediate territory along the river. It is tacitly admitted by the author that the commerce of the region, other than relates to coal, could probably be taken care of at least as cheaply by rail as by river. In any event, it seems safe to assert that, whatever may develop in the future, there exists no commercial justification for these expensive improvements at this time. Even if it were certain that Mobile would supply an adequate market for the products along the route, it may be questioned whether, when all things are considered, the commercial business of the region served could be more economically transported to that market by water than by rail. Excluding the one item of coal, experience seems to indicate conclusively that in the great region of the Mississippi Valley river transportation cannot compete successfully with rail transportation. The history of the rise, the splendid achievement, and the decadence, in the face of rail competition, of water transportation on the Mississippi River and its tributaries has

never been adequately written. Such a history would be as interesting as a fairy tale, and it would teach lessons that have an important bearing on present transportation problems. There are plenty of men yet living who saw these rivers literally alive with commerce. They can recall the time when the Ohio and the Mississippi Rivers carried great fleets of vessels of all kinds, from the rude flatboat to the gorgeous floating palace, all loaded down with the commerce of the productive and prosperous region through which they flow. The ship-yards along their banks were busy with the construction of new vessels. But this magnificent river commerce has declined until it may be said to have practically disappeared, although the actual commerce of the region has grown to many times its volume in the days when river transportation was in its prime. The railroad has taken the place of the steamboat. There must be sufficient reason, or reasons, for this great change. Commerce, in attaining its end, seeks the lines of least resistance. The decadence of this river transportation must be accepted as proof that, all things considered, the steamboat could not compete successfully with the railroad. Mr. Whinery.

Analogies drawn from the wonderful expansion and success of water transportation on the Great Lakes, are, in the opinion of the writer, totally inapplicable to ordinary river transportation. The conditions are entirely different. The Great Lakes lie mostly between parallels of latitude which also embrace the most wonderful development of human activity and commercial enterprise that the world has ever witnessed. Between the parallels of 40° and 45° north lie a chain of great cities the like of which, in their number, in their rapid growth, in their present population, and in their commercial importance, can be found nowhere else in the world. The commerce of all this region is more or less tributary to transportation on the Great Lakes. Around their western end lies the great grain-producing region of the United States and Canada, and the course of this grain to its market is parallel with their length. Along their course and toward their westerly end are situated the most wonderful iron and copper mines yet developed on the Continent. At their eastern end is found the most extensive and valuable coal region developed in the Western Hemisphere. Within the zone and in the prolongation eastwardly of the general axis of the Lakes is the commercial metropolis of North America. The region tributary to them abounds in every element necessary to stimulate agriculture, manufactures and commerce. The minerals from the the West and the coal from the East must be brought together. These lakes, with their connecting rivers, aided by comparatively inexpensive improvements, barring the ice of winter, present almost ideal conditions for inland water transportation.

For these reasons, lessons drawn from the results of channel improvement on the Great Lakes must be applied with the greatest cau-

Mr. Whinery. tion to the problems of improving our river systems. Does any one believe that if the Mississippi and the Ohio Rivers, between Pittsburg, St. Louis and New Orleans, had navigable channels that would accommodate the present vessels on the Lakes, their commerce could ever approach, much less equal, that of the Great Lakes?

It is not to be overlooked that these rivers possess now, and always have possessed, during as many months in the year as the Lakes are open for navigation, a capacity for navigation sufficient to accommodate a much greater commerce than they ever carried even in the palmiest days of river transportation. The decadence of their commerce, therefore, must be sought in causes other than their lack of navigable capacity, and he would be a bold prophet who would predict that their former supremacy could be restored by any amount of channel improvement.

If Congress could be brought to consider the question of river and harbor improvements from the cool and deliberate point of view of the practical business man, there would be a revolution in the character of our River and Harbor bills. How many members of that body, who urge and vote appropriations for many government improvement works, could be induced, were they as rich as Crœsus, to invest capital in the same projects, as private commercial enterprises, even if they could control and reap all the profits both actual and contingent? What sane business corporation, for instance, if it owned all the territory in Kentucky and Tennessee tributary to the Cumberland River, would think it wise business policy to enter upon a project to improve that river at a cost of many millions of dollars? The Government is now engaged upon such a project, involving, if the speaker's memory is correct, no less than twenty-two locks and dams between the head of navigation and Nashville. It is safe to assert that a fair interest on the cost of this project, when completed, would be more than sufficient to pay the whole of the transportation charges by rail on all the business that the improved river would ever attract.

Applying the same line of thought to the improvements in progress and contemplated on the Mobile River and its tributaries, it may well be questioned whether they will ever yield returns sufficient to justify their construction. Considering all the conditions, it is an open question whether, if the coal deposits penetrated were all owned by a giant corporation and Mobile was the chief market for the coal, it would be found more economical to make use of the improved water route, rather than of the railroads, for supplying that market. Time does not permit the discussion of this question in detail, though some of the facts have been referred to by others in the discussion of this paper.

Mr. Rafter. George W. Rafter, M. Am. Soc. C. E. (by letter).—To a student of hydrology nothing can be more interesting in Mr. McCalla's paper

than his reference to the floods in the Black Warrior River. The writer remembers, when living at Tuscaloosa, fifteen years ago, viewing with amazement the high-water mark on the piers of the bridge crossing the Warrior River at that place. It indicated that when in flood flow the stream spread over the valley, as Mr. McCalla states, for miles in width, and, for the time being, the Warrior River was, in effect, another Mississippi. Mr. Rafter.

The flood flows of this stream are undoubtedly very large, and their possible effect upon the river improvements is interesting matter for discussion.

The catchment area of the river at Tuscaloosa is given at 4 900 sq. miles, and the maximum discharge at 38.8 cu. ft. per square mile per second, thus indicating a total flood flow of about 190 000 cu. ft. per second. The minimum flow, on the contrary, of 150 cu. ft. per second, shows only about 0.03 cu. ft. per square mile per second, or the minimum flow may be taken at about $\frac{1}{1266}$ of the maximum.

The annual rainfall producing these flows is, as an average for ten years, from 1891–1900, inclusive, 48.59 ins. In Table No. 14 the writer has compiled the rainfall at Tuscaloosa for the storage period —December to May, inclusive— for the ten years, 1891–1900.

Table No. 14.—Rainfall at Tuscaloosa, Alabama, for the Storage Period, 1891–1900, Inclusive.

Year.	Annual.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	Total of Period.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1891	58.58	5.09*	8.48	10.23	9.38	1.70	1.72	36.60
1892	52.95	4.88	6.88	2.50	5.40	3.16	2.92	25.74
1893	50.00	4.42	3.08	8.27	4.41	2.61	7.10	29.89
1894	48.51	4.29	8.57	4.73	5.18	4.46	0.69	27.92
1895	51.94	5.97	7.42	2.24	7.30	2.77	4.81	30.51
1896	40.09	5.74	4.11	4.11	5.19	4.74	3.52	27.41
1897	41.46	1.11	4.43	4.83	5.63	4.07	2.71	22.78
1898	43.65	8.08	6.65	1.46	4.91	3.73	0.19	25.02
1899	50.15	3.43	8.14	3.96	7.63	2.60	2.22	27.98
1900	.....	7.88	3.58	5.54	6.08	15.67	1.43	40.18
Mean	48.59	5.09	6.13	4.79	6.11	4.55	2.73	.....

* Mean of nine years.

The figures in Table No. 14 show that the total of the storage period varied during these years from 22.78 ins., in 1897, to 40.18 ins., in 1900. Although the total in 1900 was the largest of any during the period, it does not follow that the heaviest floods would necessarily take place in that year. Flood flows are dependent on other conditions than depth of rainfall; that is to say, their intensity will very largely depend upon whether or not the ground-water is high or low at the time of the occurrence of a heavy rainfall. If the ground is full of water at

Mr. Rafter. such a time, a given rainfall will produce a much heavier flood than if the ground-water is low.

Moreover, with other conditions the same, a given rainfall will produce a larger flood during the storage period than during either the growing or replenishing periods. Taking the average of the United States, during the storage period, from 60 to 80% of the rainfall runs off; during the growing period, ordinarily only 10 to 15% runs off; while during the replenishing period, 20 to 40% runs off. These figures are very general, and may be departed from somewhat in special cases, but they serve to illustrate why extreme floods are more common in the storage period than at any other time.

The writer has defined the storage, growing and replenishing periods so frequently in his writings on hydrology that he assumes that they are understood by everybody, and hence does not repeat the matter here.

The specific application to be made of these flood data is that, with a sand and clay bottom, a very slight change in the regimen of the stream is likely to lead to extensive erosion, thus endangering the foundations of structures. Undoubtedly, this point has been attended to, and the writer merely mentions it as interesting in connection with the work on the Warrior system.

Mr. McCalla's paper is an exceedingly valuable one, and states in a forcible manner many of the important questions to be considered in the canalization of a river. One of the sections of dam proposed to be used, shown by Fig. 7, is a very satisfactory type, where an engineer builds timber dams. Other types of dams are shown by Figs. 17 and 18, but whether these will not at some time yield during an extreme flood flow can only be determined by experience. The writer cannot but think that Portland cement concrete would give more satisfactory results and prove quite as economical when the total cost for a term of years is considered; but as this phase of the question has been ably discussed by others, the writer merely refers to it at this time.

By way of showing how important it is that work be of the very best description, the following experience on the Erie Canal is pertinent: A few weeks ago, a break, involving two weeks' cessation of navigation, occurred at a culvert built in 1840. For sixty-two years this culvert has gone on doing its work, without, if the writer has been correctly informed, any repairs during the period, but suddenly, without warning, it gave way, and the masonry work of sixty-two years ago was laid bare. It appears that a bench-wall was built without any pretense of backing, with merely a veneering of thin face-stones, and how the culvert stood at all—why, in short, it did not go out when water was first let into the canal—is a mystery.

Mr. Belzner. THEODORE BELZNER, Jun. Am. Soc. C. E. (by letter).—*Notes on Sheet-Piling.*—Regarding the sheet-piles that were used at Lock and

Dam No. 4, Warrior River, which formed the inner and outer wall, Mr. Belzner. the writer believes that if a 3 x 12-in. pile had been used, made up of three pieces the full length of the pile, instead of being spliced, the results would have been better.

The sheet-piles as first ordered at Lock and Dam No. 4 were made up as follows: One piece dressed, 2 x 12 ins. and 24 ft. long, and two pieces undressed, 2 x 12 ins. and 24 ft. long. The undressed pieces were spliced, the length of the pieces being about 18 ft. and 6 ft.; one piece being spliced at one end of the pile and the other piece at the other end.

The material for the pile was any variety of pine. Wire nails ($6\frac{1}{2}$-in.) were driven through, from 1 in. to 18 ins. apart, staggered, and clinched. Three nails in a row were driven at the top and bottom of the pile and also at the splice joints.

The piles were driven with a 2 500-lb. drop-hammer, in combination with a water-jet, the jet being kept at the point of the pile all the way down. The fall of the hammer was usually 5 ft., but at times it was necessary to increase the fall on account of obstructions and hard driving.

The pile drove easily through soft material, such as soil and fine sand, and the method seems to have been efficient where the pile was to be driven through a uniform stratum, free from logs and other obstructions, but under the conditions at Lock and Dam No. 4 these piles are not efficient.

They have not sufficient thickness to prevent a tendency to buckle when driven hard. The splice, also, lessens the resistance to buckling; therefore piles made up of three pieces, 3 x 12 ins., the full length of the pile, would give more satisfactory results, and also reduce the percolation considerably.

Driving.—The writer believes that it was not the purpose of the designer to drive the pile, but to jet it into place, short blows being used to make the pile sink over the jet, but the jet would often strike a log or some other obstruction and the pile could not be made to go down by this method. In such cases the pile was driven through, or often past, the obstruction. A deflection of the point was the result, accompanied by a bending of the pile in the leads above ground. Here was shown the disadvantage of the splice, and also of the pile being of small section. The pile often had to be bent in the leads, in order that driving might continue, without breaking the pile. A stiffer pile might have forced the obstruction from its way or might have been driven through it.

WILLIAM M. HALL, M. Am. Soc. C. E. (by letter)—The writer has Mr. Hall. read this paper with much interest, and, as the literature on the locating of locks and dams is very scant, it is hoped that the Society will soon have more papers on the same subject.

The writer will not discuss the paper; but, as he has assisted in

Mr. Hall. the location of several locks and dams; and, as he has been engaged during the past year on the location of two Ohio River locks and movable dams, some of the guides which he is using in the latter work may be of interest to others. They are briefly as follows:

1. Select a location where sufficient width can be had for a lock 110 ft. wide, a navigation pass of 600 ft., and a weir of at least 250 ft.

2. Select a location which, with small expense, will admit of the passage of tows around the coffer-dams of the various parts during construction.

3. Select a location where the sill of the navigation pass can be placed above the bed of the river, and, as low as the controlling contour above and below, without danger of the movable parts, when down, becoming buried in sand.

4. Prefer a location where the channel above and below will be straight for 2 000 ft. or more; and avoid locations where it will be necessary to "flank" fleets of barges in passing through the navigation pass.

5. By use of core-drills for testing the depth to and quality of ledge rock, and by other careful investigation, select the location, filling the above conditions, which has the best natural foundation within the limits of the stretch of river on which it is possible to place the lock and dam.

6. Prefer a location at the foot of a shoal with a natural pool below and near by.

7. Prefer a location below and near navigable tributaries; thereby improving the tributaries as well as the main stream.

8. Seek locations with good, firm and high banks, and with suitable land near by for power-house, store-house and the tenders' quarters.

9. Prefer a location where the lock can be placed on the opposite shore from drift. (On the upper Ohio, the prevailing winds being from the west, unless the conformation of the river or local tributaries prevent, the wind usually carries the drift near the easterly shore.)

10. If necessary to select a location in a bend, prefer to place the lock next the concave shore, and as near to the head of the bend as other conditions will allow.

11. Avoid cutting the harbors of important towns or cities in twain by a lock and dam.

The writer wishes to acknowledge the receipt of much information on the foregoing subjects from the reading of Department Reports by W. H. Bixby, Major, U. S. A., M. Am. Soc. C. E., and B. F. Thomas, M. Am. Soc. C. E., and by the discussion of these subjects with them and with W. E. Craighill, Captain, U. S. A.

Mr. Thomas. B. F. Thomas, M. Am. Soc. C. E. (by letter).—All engineers engaged in designing and constructing works of river improvement have felt

Mr. Thomas.

the need of papers of this character, giving in detail the various parts of the design and methods of construction. Except in a general way, and in isolated pamphlets, there is no work, except De Lagrene's "Cours de Navigation Intérieure," which treats of this important branch of engineering, and this treatise, published some thirty years ago, is now out of print. Then there are numerous Government reports which contain valuable information, but a search for it involves an expenditure of much time and labor; so much, in fact, that a busy engineer cannot undertake the task. This paper, coming at a time when many engineers are engaged in the study of projects for improving rivers by canalization, is a valuable addition to literature of this class; and, owing to the care with which it has been prepared, it will prove of much benefit to such engineers, and of interest to those not thus engaged.

It is not the purpose of the writer to review the paper in detail, but rather to discuss the subject generally from the standpoint of one who has been giving it pretty close attention for a number of years.

Location.—The practice of making trial borings for foundations is a most excellent one, and no work of importance should ever be commenced without first making a careful examination of the river-bed material. This has been too often neglected, sometimes with disastrous results. Not only is this necessary in order to determine the character of the structure to be built, but it is also advisable from the standpoint of economy. Investigation, with proper appliances, of all the possible sites is the only method by which the cheapest as well as the most suitable location can be chosen. The borings, where rock exists, should extend into the ledge from 4 to 5 ft., in order to be fairly certain that a boulder or loose rock has not been encountered, and they should be made at intervals of not more than 50 ft. under the important parts of the structures. The water-jet method used by the author is very satisfactory, but, where the number of holes is limited, or where there is an insufficient depth of water to float a suitable plant of this character, the writer has found the method of driving the casing with hand pile-drivers quite economical, and sufficiently rapid for ordinary purposes. A light pile-driver, which can be carried anywhere by three or four men, is set up on the sand bars, or on a light-draft pushboat, and fitted with a banded wooden hammer of such weight as can be lifted readily. This is operated by four men pulling on a rope attached to it and passing over a sheave in the head of the driver. Two of the men stand on the second floor of the driver and two on the bottom. As the pipe is driven the material is pumped out with a sand pump, the driving being stopped during the operation. When bed-rock is reached, a churn drill is dropped into the casing, and the hole is continued with it.

In seeking his locations in "curved instead of straight reaches" the author has departed from the usual practice, but, when the curve

Mr. Thomas. is of very long radius, the idea is to be commended; because, in such locations, with the lock on the convex shore, protection from drift is afforded, which cannot always be counted on in straight reaches. It is not good practice, however, to place a lock having a fixed dam in an abrupt bend, but when the conditions require it (as they sometimes do, because it seems that the best foundations are usually on the concave sides of bends), the question at once arises as to whether to place the lock on the convex shore (the "point"), as the author has done, or on the concave bank (the "sag or bend"), as has been the most general custom. In a river carrying considerable drift, the "point" side is indicated because of the protection it affords, but such location renders a lock more difficult and dangerous of entrance from above, because of the necessity of "hugging the point" in order to keep the stern of the boat from being caught, by the stronger current away from the shore, and rounded to. Should the machinery become disabled, or high winds prevail at such a time, disaster is likely to follow, because of the proximity to the dam; and it is never wholly safe to enter from above without running out a shore line. Another argument against such a location is the fact that the quiet water or eddy immediately "under a point" causes deposit, and shallow water results, making the entrance difficult, and often requiring dredging. The danger to craft can be partially overcome by the prolongation of the land wall up stream by a retaining wall for a distance equal to the length of craft to be locked, so that boats may come alongside of it before getting too close to the dam.

Where the location is on the concave bank there is usually no difficulty of approach and entrance from above, but drift goes into the head-bay incessantly and causes more or less trouble. Owing to the current setting against the bank below, some difficulty of exit will often occur. There is much less trouble from deposit in such a location than in a point location.

Fixed or Movable Dams.—Where adjacent lands are subject to overflow it is always well to guard against increasing the height of the flood level, and it is unnecessary to state that the erection of fixed barriers across a stream which raise its low-water level will also raise its high-water level. The author gives the present depth of overflow of the valley at from 5 to 10 ft. for several miles in width. It is quite probable that the flood line will go perceptibly higher upon the completion of the system of fixed dams.

In a country not troubled with ice it would seem that movable dams could be kept up at all times, except when the stage of water was ample without them, and thus give constant navigation, and if proper care is used in construction there will be no loss of water by leakage. The writer has not yet seen a fixed wooden dam that will compare in tightness with the needle dam at Louisa, Ky., which is in a

river having a minimum discharge of less than one-third of that of the river under discussion. In the six years it has been in operation there has never been a time when the pool could not be kept at normal height. This condition, doubtless, will change to some extent, however, when others of the system have been completed, and lockages become more frequent. The author states that "the cost of construction, operation and maintenance is much greater with movable than with fixed dams." A glance at the reports of the cost of the operation and maintenance of the Kanawha Dams will disprove this supposition, and, as to first cost, it must be remembered that, while the foundations of movable dams are of masonry, those of fixed dams are usually of crib work—a cheaper and less durable material. In fifty years the wooden dam will have been partially rebuilt twice, while the masonry will still be good. Given the same grade of material and workmanship, the movable dam has no disadvantage on the score of cost of construction or in operation and maintenance. The argument that wood will last forever under water does not prove true in stationary dams, where it not only decays, but frequently actually wears out clear down to the river bed. Mr. Thomas.

The fact that tows are small will hardly justify breaking them up to pass them through the locks, because small tows are moved at much greater expense (per ton) than large ones, and the delay will add to this expense. A model river, during good stages of water, should enable a tow to proceed without interruption when once under way; under no other condition can coal be moved at a minimum of cost. As to drift carried by floods increasing the cost of operation and maintenance, experience proves that but little drift travels until after the dams are lowered. Occasionally, however, dams near large tributaries have been injured by the appearance of drift before the lowering could be completed, and it is not always wise to build such dams in preference to fixed dams.

Probably the worst enemy to movable dams is ice. It appears at times when there is not sufficient water for navigation, and yet, in order to be sure that the dams will not be injured, they must be lowered. This may strand loaded barges or other craft, and entail tremendous loss, as well as stop navigation. A case in point is the dam at Davis Island on the Ohio, which forms a harbor for Pittsburg. Owing to the early freezing of the Allegheny, it is frequently necessary to lower this dam when there is very little water for open-river navigation, and the loss and inconvenience thus occasioned are wide spread, affecting many industries in this busy community. This seems to have been a case where a fixed dam would have been preferable, even though it made it necessary that all navigation should pass through the lock. Another solution, and possibly a more satisfactory one, would have been the construction of a bridge dam (which

Mr. Thomas. can be operated in the ice) at the site of the next dam below, which is fortunately in a narrow place in the river where there is an island. The cost of such a dam probably would not greatly exceed that of the two which it would replace, and there would be only one lock to pass through, instead of two as at present, a very great advantage on that river, where the tows are large and numerous.

Size and Depth of Lock.—One of the experiences quite common on improved streams is to find that insufficient depth has been provided, and that the locks are too small. Usually, the depth is regulated by the draft of the largest craft when loaded, with a slight allowance for clearance, but it seems that no sooner have good depths been provided than better depths are demanded. Thus, on the Upper Ohio, a start was made a few years ago with 6 ft. on the sills, and 8 ft. is now asked for. Another example, still more striking, from the fact that the demands were granted, is the improvement of the Seine below Paris. The original slack-water system was established between 1838 and 1853, at a cost of \$2 800 000, with a depth of 5¼ ft. This depth was increased, between 1858 and 1878, to 6½ ft., at an additional cost of \$2 800 000. Ten years later the depth was increased to 10½ ft., at a further cost of \$12 200 000. The commerce, which amounted to very little under the first arrangement, owing to the inroads made upon it by the improvements in railroad traffic, has increased enormously with the greater depths for navigation. Similarly, may be cited, the St. Mary's Falls Canal improvements, completed in 1855, at a cost of \$1 000 000. Between 1870 and 1881 the additional sum of \$2 000 000 was expended for the purpose of accommodating the increasing commerce. In 1896 a lock, 800 ft. long and 100 ft. wide, giving a depth of 21 ft., was completed at a cost of \$3 700 000. That further improvements will be required before many years, is quite apparent to those who are conversant with the commerce passing through the canal. It is evident, then, in studying the question of depths to be given on a river which is at all worthy of improvement, that it is far better to provide a much greater draft than indicated for present needs, in order to be reasonably certain that future commerce may be well served. The same idea should also be applied in fixing the size of a lock, because the trend of modern transportation is toward cheap rates, and these can be had only by using larger craft. It is much less expensive to build beyond present requirements in the start than it is to remodel later on to suit new conditions. The lift to be given a dam is determined by the rights of riparians, the proximity of water mills in tributaries, the heights of bridges crossing the stream, and by local conditions, such as suitable foundations, etc. Considering navigation interests solely, the greater the lift the better, because then the number of locks will be reduced, and boats will make better time and be subjected to fewer dangers. As a general thing, movable dams

Mr. Thomas.

are much lower than they should be, and in many localities fixed dams are open to the same objection, but, as a rule, these have about all the height allowable, and in some places their effects on the unprotected banks below are very injurious. Unless the banks are of hard material they should be well protected below all fixed dams and particularly those of high lift. For movable dams, this protection can be much less, and is usually not necessary on the lock side, owing to the fact that the dams are lowered from the abutment end. In many instances the bank protection can take place gradually, as the original material is eroded.

Materials of Construction.—A good many of the older locks were built of wood; then came stone; and now it is concrete, which has not yet been in use on this class of work long enough for engineers to be certain of its durability. No reason is apparent, however, why it should not be all that is desired, and it certainly simplifies construction very greatly, particularly in localities where good building stone is inaccessible. At first there was a disposition to use natural cements, but, as the Portlands became cheaper and better understood, they displaced the former, until now nearly all these structures are being built of American Portland cement. The prices have gone down, as the number of manufactories increased, until, within the past year, it has been possible to procure a first-class Portland cement in the Ohio Valley for less than $1.50 per barrel. The better grades of natural cement have proved quite satisfactory under water, but there are so many poor grades manufactured and sold that they have prejudiced the use of natural cement altogether. At first there was a disposition to criticise the appearance of concrete work, but, after several years' use, it is believed that this objection will not be apparent, as all walls become more or less marred by the rubbing of boats and barges. It is thought, also, that Portland cement will be too hard to show many scratches, and that, after ten years' service, the appearance of a lock face built of it will be superior to that of stone heretofore used. There is no doubt of its being a cheaper material in almost all localities where locks are to be built, and it eliminates entirely "stone cutters' strikes" and the use of skilled labor. Of late years this is a very considerable pecuniary advantage, because of the eight-hour law, which requires the United States to pay the same for eight hours' work as is paid for a full day's work by others in the locality. The rapidity with which a concrete wall can be put up is one of its chief advantages, as on many rivers the working season is very short, owing to high water; and it is impracticable to do very satisfactory stone work at night, as each stone must be brought from a pile at some distance and swung into place and lined up. With concrete the work can proceed without stopping, and one mixer will put out from 400 to 500 bbls. of cement in 24 hours, barring breakdowns and other delays. A single shift,

Mr. Thomas. working in daylight only, is placing an average of 200 cu. yds. of concrete daily, to the writer's knowledge. The proportions used are 1 part of cement, 3 of sand, and 6 of broken sandstone. At another lock the proportions are 1 bbl. of cement, 15 cu. ft. of sand, and 33½ cu. ft. of broken limestone, varying in diameter from ½ in. to 2½ ins. This appears to be a little "lean," but the product seems to be hard and strong. Common river sand is used, about 70% of which will pass a No. 30 sieve, 50% a No. 50, and 2% a No. 100. Sandstone is somewhat soft, crushing slightly during the process of ramming, but limestone is very satisfactory. Experience proves that pieces which will barely pass through a 2½-in. ring are too large, and that better results can be obtained by reducing this to 1½ ins. and taking all the smaller particles down to ⅛ in.—virtually, "run of crusher" stone. The larger stones seem to bunch together and cause voids, and will not pack like the smaller ones.

The author has adopted the block method of building; that is, each day's work forms a separate monolith. The method in most general use, probably, is that of the continuous monolith, in which the concrete is built from bottom to top without stopping. The block method is used on the Kentucky and Big Sandy Rivers, and is gradually growing in favor, although a stronger and more presentable wall can doubtless be made by the other method. In the continuous-monolith construction the form timbers are put up the entire height to begin with, and it is necessary to lift the concrete to the top and then lower it to position. This is frequently quite troublesome, the buckets knocking the forms out of line and threatening the safety of the men cooped up underneath. When the concrete is built in blocks it is only necessary to carry up the forms as the work progresses, and night work is not required at all. The design of forms varies in different localities, the most usual type being that with posts erected, inside of which lagging is placed as the work proceeds. The posts are braced at intervals, as may be necessary. Another style is that in which the lagging is tied together with rods, and held apart by braces, the rods being generally left in the completed work. There is some difficulty in holding forms of this class exactly in line, and their use is not general, although it effects quite a saving in cost. Dressed lumber should always be used for lagging, on all faces to be exposed after the completion; and a lining of sheet iron would prevent the graining of the wood from showing and would otherwise improve the appearance of the face of the wall. However, after a few years' use all chamber faces on rivers having a good commerce show more or less injury, and it is not essential that a beautiful wall be had at the start. The forms can be removed in from three to five days and used again. Upon their removal it is a good idea to go over the face of the wall and rub down rough places with a block of wood or soft stone. This will

improve the appearance of the work greatly, and can be done very readily if attended to immediately after the lagging comes off. Mr. Thomas.

Gates, Valves, etc.—The gates seem to be very well designed. Steel is now replacing wood for such structures, and will eventually do so entirely, unless experience proves its inadaptability for such work. It sometimes happens that boats strike the gates with considerable violence. In a wooden gate this does not result in permanent injury or in injury which cannot be repaired then and there. With a metal gate, the case is different. If it is struck with sufficient force to bend it, it may not spring back to place, in which case it will not miter, and becomes useless. To repair the damage it may be necessary to remove the gate and transport it to a shop—quite an undertaking. There are two general types of metal miter-gates now in use and under construction on the rivers of this country: In one class the beams are placed horizontally, and transmit the strains to the walls and the opposite leaf through heel and toe posts; in the other class the supporting beams are placed vertically, connecting with horizontal girders at bottom and top. This class of gate has less metal than the other when the length is considerably greater than the height, but a heavy strain is transmitted through the top girder to the wall well toward its top. In the writer's opinion it is not to be preferred to the horizontal framed gate. A more serious objection to it is that when rust has weakened the uprights at the water line, as it probably will, the entire gate must be renewed. With horizontal framed gates it would only be necessary to renew the lower portion. The use of a buckle-plate skin will permit of a wider spacing of girders and effect a material saving of metal, because these plates are much stronger than flat plates of equal weight, and will require no stiffeners, even in very wide panels.

Originally, metal gates had a double skin in order to secure buoyancy and consequent reduction of weight, but it was found that the compartments leaked and were not kept pumped out, so that the additional plating was simply so much weight to be carried without any beneficial result. It has also been found that the upper compartments fill with mud and induce rust. Without doubt the single-skin gate is to be preferred in every way. One objection to all metal gates is that they are likely to rust at the water line and below, when they cannot be kept clean and painted, and the writer has considered the advisability of building the upper portion of metal and the lower of wood. The latter could be renewed at any time without disturbing the former, and with very little delay to navigation. Wooden gates are short lived. The writer knows of few gates which have lasted more than sixteen years; many are partially renewed within ten years, and a few in less. There is only one lock provided with iron gates on the tributaries of the Upper Ohio, but these seem to be perfectly good after

Mr. Thomas. twenty-one years' service. In his work on mitering lock gates, Major Hodges gives the average life of a metal gate at forty years, when properly cared for and repaired.

The method of operating the gates adopted by the author seems to be undesirable, although experience may prove otherwise. Mechanism somewhat similar, but, doubtless, imperfect in construction, was tried a number of years ago on a lock on the Kentucky River, and, later, was supplanted by something else. The writer was told that it was not satisfactory, but has no information on the subject. The most common method of gate operation is by a rack-bar and pinion turned by capstan bars, but it is not wholly satisfactory, partially because of the friction of the rack and pinion and partially because the bar, in order to reduce its length, is attached to the gate too far from the toe. The primitive wooden spar and modern wire rope winding on a capstan, as seen on the Kanawha and elsewhere, is the simplest and most satisfactory device the writer has yet observed for hand operation. The spar and line are attached at the toe, the length of spar not being objectionable when it is an ordinary pole, and the best point from which to operate is thus secured. It is worthy of note that the new locks on the Moldau, in Bohemia, are provided with rack-bars operating under the coping, where they are out of the way during maneuvers. The same system has an example on the Fox River in this country.

The cost of wooden gates, for locks from 50 to 55 ft. wide, with about 12-ft. lift, ranges from $1 500 to $2 500 per leaf, usually being about $2 000. Steel gates will not usually cost any more, if designed with a view to economy. In adopting the balanced valve the author followed the practice which has the advantage of long and fairly satisfactory trial, and, with the modifications and arrangements which he has provided, most excellent results should follow. This valve is sometimes used with a vertical and sometimes with a horizontal axle, and seems to work equally well in either way. Other forms of valves in use are the gridiron sliding valve, in which the valve is pierced with rectangular openings, and the cylindrical valve, which consists of a cylinder, from 4 to 6 ft. in diameter and 18 to 20 ins. high, sliding vertically inside a chamber having a water-tight cover. This cylinder or ring encircles the culvert opening, and, when down, shuts off the water. When raised, which is an easy operation, as the water is pressing equally from all sides, the water goes into the culvert under the cylinder. This valve is in satisfactory use on the Muskingum River, and is being adopted for new locks on the Big Sandy and Kentucky. The writer has been considering the advisability of dispensing with the chamber over the cylinder, dropping its cover down to the top of the cylinder, and sliding the latter up on guides on the outside. This cylindrical valve has been rendered self-operating by the introduction of water through its cover, and a patent has been issued to

Major W. L. Marshall, and another to Mr. Sanford L. Cluett, for devices of this character. Some of the latter have been made for trial on the Big Sandy, but are not yet in use. Mr. Thomas.

General Remarks.—In preparing plans for work of the character of that under discussion it should always be remembered that repairs will be costly, tedious and detrimental to navigation, because the water will be troublesome, and it will be necessary to close the locks for a time. It is of the first importance, then, not only to make the various parts as simple as possible, but also to make them stronger than necessary. Especial attention should be given to designing all valves or other movable parts so that they can be readily removed for examination or repair. It is only necessary to go over one of the older improvements to be convinced of the necessity for such cautions. The exposed ends of anchor bolts will rust eventually, so that renewal will be necessary, and it will be well to provide sleeve nuts in the original construction so as not to necessitate digging the old bolts out of the masonry.

Of late years, considerable taste has been displayed in the design of lock buildings and in the care of the grounds. These places should be veritable parks, with grass, shade, flowers, and neat walks and fences; in fact, they should be object lessons to their several communities. A very little money will go a long way when all the labor is ready at hand and under pay the year round. The works themselves should be kept in the best of repair, and lock gates, irons, machinery, boats, and such property as can be improved thereby, should be thoroughly overhauled and painted every year. It is a lesson, to those engaged upon the works, which is very cheap at the price, as it causes each man to be more careful, to take more pride in his work and in himself, and thus he becomes a valuable citizen as well as a valuable employee.

D. M. Andrews, M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution on a subject of which there is far too little information available. There is little to criticize in the paper; the writer, however, wishes to discuss certain of the items from the viewpoint of his own experience. Mr. Andrews.

The Black Warrior and Warrior are one and the same river; the name Black Warrior changing to Warrior at Tuscaloosa, as explained by the author. The Black Warrior has stable banks and bottom. The Warrior has caving banks and a shifting bottom.

The adoption of the fixed type of dam for the Black Warrior was unquestionably the best; and, for the reasons given by Mr. McCalla, particularly the small low-water discharge, the fixed type was the only type that could have been safely adopted for the Warrior. With the fixed type, however, the pools between the dams will in time fill up, and constant dredging will be required to maintain a channel of the

Mr. Andrews. proposed width and depth between the locks. With dams of the movable type this filling of the pools could be prevented by lowering the dams during periods of high water, and letting the river clear itself of accumulations of silt and gravel.

The writer believes that the advantages or disadvantages of locations at convex or concave shores are, except in abnormal situations, more apparent than real; however, other things being equal, a straight reach is probably the best location. The writer's experience leads him to the following conclusions:

There should be guide piers, both above and below every lock, placed an angle of about 10° with the axis of the lock. Where there is much disturbance below the dam during periods of high water, the lower pier should extend from the lower end of the river wall. Solid piers not less than 150 ft. in length, and longer where the volume of traffic is considerable, with drift passages at the lock wall, are preferable to pile clusters.

Rack-bars for maneuvering lock gates are objectionable in rivers carrying much drift, for they have to be removed when the lock is about to be submerged, and replaced when it emerges. The maneuvering apparatus described by the author is not open to this objection, and, as the writer knows from experience, is in other respects a most admirable arrangement.

The design of the dams at Locks Nos. 1, 2 and 3, Black Warrior River, is bad, on account of the stepped lower slope. Drift passing over the dams would cause the displacement of the slope stones, and entail constant expense for repairs. A better arrangement is that in which the stones are set with the beds normal to the slope, as at Lock No. 4 of the same river. Dams of this type, *viz.*, crib-filled dams with a lower slope of stone should have the lower slope laid without cement beds or joints, thereby allowing the free passage of leakage. The face, whether of masonry or sheet-piles, should be made water-tight, or as nearly so as possible; but in rivers of considerable low-water discharge, it is not the part of economy to go to any great expense to secure a water-tight face. As usually designed, it is only necessary to maintain the surface of the pool at the crest of such dams. The low-water discharge not needed for that purpose may pass the dam as leakage. This last statement does not apply, of course, to dams intended to develop power.

The writer has recently repaired and raised to a height of 15 ft. (the height being originally 12 ft.), a crib-filled dam, 700 ft. long, built on a foundation of gravel, mud and rotten limestone. The repairs and addition to the height were made by building a heavy stone slope from below and over the crib, or what was left of it. The stones in the slope contained from 1 to 3 cu. yds. each, and were laid dry with the beds normal and the faces flush with the slope. The face of the dam

was of double-lap, 4 x 12-in. and 2 x 12-in. sheet-piling. The sheet-piling extended 5 ft. above the crest, as a protection bulkhead to the workmen below. After the slope below was finished, the piling was cut flush with the crest. The operation was repeated until the dam was finished. Quarry waste was filled in against the sheet-piling from above. No attempt was made to get a water-tight face, the low-water flow being sufficient to maintain the upper pool at the crest, with the leakage allowed. An apron at the toe, to prevent scour, was built of large stones roughly placed, and the interstices were filled with smaller stones. The work was done by hired labor, and cost, every expense included, \$2.51 + per cubic yard. Mr. Andrews.

As to the expediency of the improvement of our rivers, opinion is as diverse as it once was concerning the proper gauge for our railroads, and it seems that the arguments for and against are likely to be as vigorously presented as were those fought over in the days of the "Battle of the Gauges."

The writer can do no more than generalize, not having statistics at hand, but he ventures the assertion that, should the Erie and Welland Canals be abandoned, freight rates from the West to New York would go soaring, and New Orleans would become the chief shipping point for the export trade of the country. He is aware that pages upon pages of statistics have been printed showing that the railroads entering New York from the West can and do carry freight as cheaply as the same is transported through the canals; but those statistics being presentations of facts make no prediction of the probable effect of an abandonment of the water lines of transportation from the Great Lakes to the Eastern Coast.

The writer recently had some machinery shipped to a point on the Lower Mississippi River, a part coming from Bridgeport, Conn., and a part from Birmingham, Ala. The freight rate from Bridgeport was practically the same as that from Birmingham, because Bridgeport, though the length of haul was against her, had the advantage of continuous water transportation.

Statistics of this kind interest the people at large, and, as long as conditions remain as they are, there will be a demand for the improvement of our rivers, though argument be piled upon argument showing that railroads can be built, operated and maintained, at far less cost than the cost of improving, and maintaining the improvement of, our rivers.

NAT. A. YUILLE, Esq. (by letter).—In his very interesting paper, Mr. McCalla mentions the borings for the foundations, with which the writer was intimately associated for several years. A description of some of the details of this work may be interesting. Mr. Yuille.

The Black Warrior River varies in width from about 500 to 1 500 ft., and is confined by high, steep banks, and, in many places, by precipi-

Mr. Yuille. tous sandstone cliffs, several hundred feet high. At frequent intervals, and extending entirely across the river, are natural dams of bed-rock. It was proposed that the locks and dams in this river be built of sandstone masonry upon this bed-rock foundation.

The oscillation of this river is considerably less than that of the Warrior, and becomes less and less in proceeding toward the head. By means of coffer-dams the construction of the locks can be continued throughout the year, and would not be interfered with, materially, by many rises in the river that would make construction impracticable in the Warrior River. At Lock No. 4, on the Black Warrior River, the existence of bed-rock suitable for a foundation was known, the borings being taken to ascertain the kind of material to be excavated, and the relative amounts of earth and rock excavation. The estimates were based upon the result of these borings.

The Warrior River varies in width from about 500 to 200 ft. or less. Except in a few places, the banks are alluvial deposits which during high water are flooded from 3 to 10 ft., the river widening from 3 to 5 miles. There are numerous shoals, bars, and a few rock reefs in the Warrior, but none of the reefs is of the good sandstone so plentiful in the river above. In fact, with the exception of a very poor quality of sandstone seen in several places in the vicinity of Lock No. 6, there is none cropping out below Tuscaloosa. A short distance above the final location of Lock No. 6, are two rock reefs, upon one of which it was at first thought the lock would be built; but examination by boring developed the fact that the reefs were only thin crusts of very poor sandstone, the principal cementing substance in which was iron oxide. Most of the other reefs examined along the river were of soft, rotten limestone, which usually extended only part way across the river, and then dipped too far below the grade of the lock to be used without some piling. This rotten limestone, or marl, under the crust or under water was soft, like very hard, stiff clay, and of deep, grayish-blue color. When exposed to the atmosphere and sunlight it became hard and brittle, and of a pale blue color—almost white.

The hope of finding safe natural rock foundations being abandoned, locations avoiding these reefs and strata of rock were sought, where piling could be used under the entire structure. At Lock No. 3 considerable difficulty was encountered in avoiding several thin strata of rock, upon which it was considered unsafe to build, and which were too thick to be penetrated by piling. A distance of 3 miles was examined thoroughly before deciding upon the final location. Fig. 3 shows the plan and profile of the borings and the position of the rock at the final location of this lock.

Perhaps the greatest difficulty encountered in deciding upon the location of any of the locks was at Lock No. 2, Warrior River, where fourteen possible locations within a distance of 5 miles were examined;

Mr. Yuille.

the objection in each case being the presence of soft, rotten limestone above the foundation of the lock on one side, while on the other side the rock dropped too deep to build upon without piling. It was not considered advisable to build part of the structure on rock foundation and part on pile foundation, for fear of unequal settlement.

While the greatest difficulty in locating a lock site was at Lock No. 2, Warrior River, the greatest difficulty in the actual borings was at Lock No. 6, where the immense bed of clean gravel penetrated made the taking of borings very laborious, slow and expensive.

In connection with a topographical survey of the Warrior River, made by the writer in 1898, probings were taken at frequent intervals. These probings were taken by two men working an iron rod down into the ground by hand; the idea being to locate rock near enough to the surface to use it for foundations. At the same time this survey was being made, another party was in the field making a more thorough examination, by taking probings in squares of 10 ft. under the locks, as projected. A 1-in. iron pipe, with a solid point screwed in the end, was used in this work and was driven with a small, hand pile-driver, usually obtaining greater penetration than in those made by hand rods. In connection with these probings, two water-jet borings under the lock and one under the abutment of the dam were also taken. The water-jet borings were practically the same as will be described later, the details of the apparatus and the methods of taking them being changed constantly as suggested by experience. Subsequent borings and excavations have proved these probings to be very unsatisfactory; in fact, they were misleading, as erroneous inferences were drawn. The information obtained was negative rather than positive. It became known that there was no rock above the bottom of the probing, but as to what stopped the iron rod—whether it was solid rock, or merely a piece of gravel, or accumulating friction—could not be known. When it was decided to abandon search for rock reefs, it was also decided to discontinue these probings and take water-jet borings only.

During the following year a survey and boring party, of about twenty-three men, was put in the field, with the writer in charge. They lived in a quarter-boat and in tents on the bank. Two gangs, of six men each, operated two sets of boring apparatus, and a third gang of five men pulled the casings. The other six men did the necessary surveying, locating borings, cross-sectioning the site of the work, laying out property lines, placing stone monuments, etc. The previous topographical survey assisted greatly in the work of selecting lock sites, all possible locations being selected from the map and profile, and each one being tested by further surveys and trial borings.

Boring Apparatus.—A 2-in. casing and a 1-in. water pipe, both in

Mr. Yuille. sections about 10 ft. long, were used. Two light four-legged frames, with loose boards, adjustable at different heights, for the men to stand on while operating the apparatus, were used. A heavy tripod, with a hand-winch attached, and with two detachable wheels for transporting it, was used for pulling the casing when considerable power was required. With this machine the threads of the casing couplings could be stripped. A similar tripod, of lighter pattern, with double blocks and falls, was used for easy pulling.

Heavy wooden mauls, 10 ins. in diameter and 7 ft. long, operated by four men, were used in driving the casing. These mauls were lifted vertically, by means of wooden bars passing through auger holes in the upper end, and dropping upon the casing. In driving, the pipe was protected by solid iron heads, 12 ins. long, screwed over the end. These driving heads were also used in pulling the pipe, and were threaded in the socket so as to fit both 2-in and 1-in. pipe. Three different kinds of drill bits, all perforated with water holes, were used for rock, clay and gravel. The gravel bit was most generally used. In cases where the 1-in. water pipe broke, the bottom part was sometimes lost and sometimes recovered. The best device for recovering a pipe was a piece of tapering, barbed iron screwed into a piece of pipe. The barbed iron was jammed into the lost pipe, which was then drawn up.

Two small Cameron steam pumps, No. 0 and No. 1, furnished with steam from a 6-H.-P. vertical boiler, were used for forcing water into the borings. When the distance was short, 1-in. Eureka mill hose was used from the pumps to the borings. When the distance was greater, 1-in. pipe and the hose were both used.

Method of Work.—In boring, the drill-pipe is churned up and down by two men using a T-pipe handle at the top, to which the water hose is connected. Each time the drill-bit lands, the pipe is given a quarter turn to the right, to cut the material at the bottom and to keep the couplings tight. Generally, the drill-pipe is kept a few inches below the casing in order to form a cavity at the bottom, and thus facilitate the driving of the casing. In both driving and pulling, the casing should be twisted frequently, by means of a lever clamped to it, in order to keep it free, and it should always be twisted to the right to keep the couplings tight.

When necessary to drive the casing, the upper length of drill-pipe is taken off and the driving head screwed to the next length. This is then lowered and the same driving head is screwed to the top of the casing. The casing is then driven, the drill-pipe being suspended inside, with the drill-bit about 10 ft. above the bottom of the casing, and thus protected from injury. When the casing is driven as far as it will go, the driving head is taken off, another length of drill-pipe is added, the drill-pipe is lowered and boring resumed.

A T-spout is kept near the top of the casing, and through this the materials washed up pour out. Samples are taken in a cup constantly, and changes of material at once noted and the length of pipe measured. Thus the thickness and elevation of the different strata are obtained and recorded. The different materials are classified as to composition, fineness, hardness, compactness and color, especial pains being taken to make the classifications as uniform as practicable. Under the heading of remarks should be noted other items, such as fossils in the rock, embedded logs, artesian water, time of boring through a certain thickness of hard strata; also causes of delays, mishaps, etc. Samples of some of the strata were bottled, labeled and placed on file. Mr. Yuille.

Borings were taken under the sites of all structures and through the approaches at intervals of about 50 feet. Generally, the borings extended at least 30 ft. below the lock floor, and one boring at each lock site extended at least 80 ft. below the lock floor. The deepest boring made, 188 ft., was at Lock No. 2. The 200 ft. of drill-pipe used for this boring was handled by six men with some difficulty. Borings taken to show the character of excavation, as a rule, extended only a few feet below grade.

The greatest difficulty occurred in boring through thick beds of coarse gravel; all the pump water sometimes percolating through the gravel instead of coming up through the casing. The best method of getting through such gravel is to drive the casing as far as possible, and, with the drill-bit, crush and grind the lumps of gravel confined within the casing. Then the water which does not escape from the bottom of the casing will bring up the small particles of broken gravel. The drill cannot work below the casing in such gravel, as all the pump water will escape, and coarse gravel will fall in above the drill-bit and jam it. For one casing, $2\frac{1}{2}$-in. pipe was tried, with 1-in. drill-pipe, but it required so much more water to give the necessary upward velocity in the larger annular space between the two pipes, and the friction on the outside of the casing was so much greater that, instead of diminishing the difficulties encountered with the 2-in. casing, it increased them greatly.

Sometimes it was necessary to continue the borings through and below strata of rock through which the casing could not be driven. In such cases, if soft material were encountered under the rock, the classification became unreliable, because materials from different elevations in the uncased hole kept falling in and coming to the top of the casing, there being no way of ascertaining just whence they came.

Cost.—For Locks Nos. 1, 2 and 3, Warrior River, 86 trial borings, aggregating 3 841 ft. in depth, were made at a cost of \$1 676, or $43\frac{6}{10}$ cents per foot; 236 final borings, aggregating 9 875 ft. in depth, were made at a cost of \$2 185, or $22\frac{1}{10}$ cents per foot. The greater cost of trial borings per foot was largely due to the delay in moving from

Mr. Yuille. place to place. The cost of survey work, in connection with the borings at the three lock sites, was about $700.

Mr. McCalla. R. C. McCalla, M. Am. Soc. C. E. (by letter).—The writer feels gratified that his paper has brought out an able and interesting discussion of a subject on which the literature is rather scanty, considering the number of important works of this character in the United States and other countries. He hopes to profit by many of the ideas and arguments advanced.

Location.—The Warrior is a narrow, crooked stream. There are very few straight reaches and few or no places wide enough to take in the lock and abutment and a dam long enough to provide sufficient spillway for the proper discharge of flood waters. The width of the stream at low water varies from 100 to 300 ft. and averages about 200 ft., therefore the locks and abutments must be set well back into the banks. If straight reaches were available for locations, curved approaches leading back into the stream would be required at both ends of the lock. If locations were made in bends and on the concave shore, still sharper or longer curves would be required in the approaches to get back into the stream.

Wherever a lock is located, it forms a barrier to the current, and thereby causes silt deposits in the lock and approaches; also, during rises there must be a strong off-shore current above the lock toward the "suck" of the dam. The deposit and off-shore current are both somewhat worse on a convex than a concave shore. With locks in the bends and on the convex shore, straight approaches parallel to the axis of the lock are secured, and they re-enter the stream within a short distance.

During about eight months each year the pools are like ponds, without current; and, certainly, during that period, straight approaches would be better for navigation. At times during the other four months, wherever the locks are located, it will be dangerous to enter from above without putting a line ashore, and snubbing posts will be placed along the banks for this purpose.

Vast quantities of drift are borne by floods. At times the writer has seen the drift so thick that it looked almost as though one could walk across the river on it. This drift is constantly becoming waterlogged and dropping to the bottom in eddy water. Concave shore locations would place the locks right in the path of this drift, and would cause much of it to accumulate in the locks during floods. It will probably be easier to pump out silt than to remove a combination of silt and drift.

All things considered, the writer believes that, for this system of rivers at least, a convex shore in a bend of long radius gives the best location for a lock.

Dams, Lift, Guard, Etc.—The question of fixed or movable dams admits of many arguments on both sides. Movable dams are at best

rather frail structures. Tows on the Warrior will always be small, and will require from one to two lockages, only, to pass a lock. Movable dams would have to be kept up about nine months per annum, and would have to be raised occasionally during the other three months. Therefore, to navigation, the saving of time from the use of movable instead of fixed dams would be small. Probably no form of movable dam except the needle-dam is sufficiently tight to hold the low-water flow of the Warrior; and a serious objection to the needle-dam is that the needles must be taken out of the river and stored when the dam is lowered, and brought back when the dam is to be raised. Mr. McCalla.

The Warrior and Tombigbee are both quite flat, and all the indications are that fixed dams of 10 ft. lift and proper length will drown quickly during floods and exert little or no influence on maximum flood heights. The dam at Warrior Lock No. 6 was completed during September, 1902, and a small rise occurred during the following month. Daily gauge readings at 7 A. M. during the rise are shown in Table No. 15, the zero of both gauges being the lower miter-sill, Elevation = 74.5, and the crest of the dam being 16.5 ft. above zero, or Elevation = 91.0:

TABLE No. 15.

Date.	Above the dam.	Below the dam.
Oct. 11	18.51	9.00
" 12	20.25	15.00
" 13	20.60	17.20
" 14	19.70	14.20
" 15	18.85	11.20
" 16	18.29	9.20
" 17	17.99	7.90
" 18	17.70	7.05
" 19	17.60	6.30
" 20	17.50	5.87

Dam No. 5, seventeen miles below, the crest elevation of which is 81.0, was nearly completed at the time. Part of the rise went over this dam and part through the chamber of Lock No. 5, where the gates had not been erected. It will be noticed that the lower pool gained quite rapidly on the upper pool at Lock No. 6, and, had the rise continued, the two pools would probably have reached approximately the same level several feet below the top of the lock walls. Of course, on a more rapid rise, the lower pool would not have gained so rapidly on the upper pool, but it is thought that the maximum difference in pool levels when the lock walls become submerged will be about one foot, and that this will occur very rarely.

In the Warrior River the principal objections to concrete dams on pile foundations are the cost and the danger of "blow outs" underneath. It is believed that the stone filling in the crib dams, resting directly on the river bed, will settle and choke a leak before it can

Mr. McCalla. become large, and that thus the danger of "blow outs" is reduced to a minimum.

The object of caulking the sloping sheathing is to prevent streams of water from trickling through the stone filling, carrying the sand and gravel out of the interstices, and thus reducing the weight of the filling. Weep holes are provided in and near the down-stream face to relieve the sheathing of dangerous up-thrust. So far, there is apparently very little leakage through or under Dams Nos. 5 and 6, which are the only ones of this type completed on these rivers.

Scour is expected below the dams during floods, and it is to be guarded against by heavily rip-rapping the bed of the stream below the apron and along the back of the river wall and the face of the abutment.

Sheet-Piling.—Sheet-piling 6 ins. thick was found to be rather light for hard driving, and on future work 9-in. piling is to be used almost entirely. Each pile should be built of three pieces extending the full length of the pile, and the center pieces should be dressed to uniform thickness to reduce leakage. The best width of tongue for piles built of 3 x 12-in. planks seems to be about 3½ ins., and great care should be taken not to injure the piles by hard driving, the combination of jet and short blows of a heavy hammer giving the best results. The heads of all piles should be protected by a Casgrain pile cap during driving.

Pumping Out.—Thus far, there has been no difficulty in pumping out the lock chambers with the coffer timbers in place, an 8-in. centrifugal pump being ample to control the leakage under ordinary conditions.

Sill Anchorage.—The miter-sills are held down by 1½-in. bolts, 4½ ft. long, and spaced 3 ft. apart. Each bolt has an 8-in. cast washer on the bottom and a 4-in. wrought washer and nut on the top. The sills can hardly come up unless the walls come with them.

Maneuvering Gears.—The gate-maneuvering gears have been in constant use seven years on the Tuscaloosa locks, and have proved very satisfactory. A small boy can operate the gates with ease under ordinary conditions. The gears are simple, durable and efficient. They lie flat and close to the walls, and nothing has to be removed during floods except the lever, which is unshipped and laid down by the gears. The writer believes that this method is the most satisfactory now in use for the hand maneuvering of lock gates, and wishes to give full credit for the design to the late Horace Harding, M. Am. Soc. C. E., who was in local charge of these improvements for many years.

Stability of Walls.—The river wall is 6 ft. wide on top, 12.15 ft. wide at the base, and 29.5 ft. high, but rests on a concrete footing course, 14.25 ft. wide and 1.5 ft. thick, with a tight mortar joint between the base of the wall and the top of the footing course. Up-thrust

in this mortar joint is not considered possible, and, therefore, the wall proper has a factor of safety, against overturning on the footing course, of about 6, with 17 ft. head outside and the lock chamber pumped out. Mr. McCalla.

Up-thrust undoubtedly exists under the footing course, but the head must be reduced considerably while the water is percolating through or under the sheet-piling and through the sand and gravel underneath the wall on its way to the floor valves. Assuming an up-thrust due to half the maximum head distributed uniformly under the footing course, the factor of safety against overturning would be about 2.5, with 17 ft. head outside and the lock chamber pumped out.

With the water level in Warrior Lock No. 6, at Elevation 99, the water level in the lower pool should be about at Elevation 96 to 97, instead of 93.8, as assumed by Mr. Nelles. The gauge readings given in Table No. 15 illustrate the rapid gain of the lower on the upper pool level during rises.

The bank walls are believed to be perfectly safe under the worst possible conditions. With the lock chambers pumped out, fresh filling was placed behind the bank walls level with the top, and was thoroughly saturated with water to assist settlement, the water standing in pools for days at or near the coping level. No evidence of settlement or movement of the walls can be detected. The bank walls are 6 ft. wide on top, stepping at the back to 16 ft. wide at the base, with steps 2 ft. wide and 5 ft. high. The footing course is 18 ft. wide and 1.5 ft. thick, projecting 1 ft. on the front and back. Both walls are 31 ft. high, including the footing course.

Guide Walls, Masonry Floors, Etc.—Long guide walls above and below the lock bank wall, masonry lock floors, masonry dams (if they can be secured against "blow outs"), and other features of this character suggested by those discussing the paper, are excellent things, but they cost a great deal of money. If all these suggestions were adopted for these improvements, the first cost would be perhaps $10 000 000 instead of $5 000 000, and the time required for completion perhaps would be doubled also, because the biennial appropriations for a particular improvement are necessarily limited by the needs of the whole country and the funds available. At the present rate, eight to ten years will be required to complete the work, while the traffic is nearly all prospective, and, possibly, may never develop.

Under these conditions it is deemed to be to the best interests of the United States to complete the work as rapidly as practicable, and at the minimum cost. Betterments can be made as the traffic justifies them and as the old works wear out. Permanent guide walls of proper height and length would be very costly and would not be justified for a small and unimportant traffic. It would be better to let the boats put a line ashore occasionally. Temporary guide walls of timber cribs, or piles and timber, if built now, would rot down before the

Mr. McCalla. lock system is completed, and, therefore, before there can be much traffic. If temporary works of this character are to be used they should be built at about the time of completion of the permanent works. Besides, after the locks have been in use a while, the character of the guide walls, guard protection, etc., best suited to the needs of traffic and to local conditions can be determined better than now, for experience is an excellent teacher.

If the timber crib dams prove safe and satisfactory, but finally rot or wear out, they can be replaced with concrete dams on the old foundations, which should be thoroughly settled by that time. If the timber floors fail or wear out rapidly they will have to be replaced with more permanent construction.

If our pioneer railroad builders had adopted the policy of using nothing but the best and most permanent construction, nine-tenths of the United States would still be a wilderness. On the contrary, they built cheap lines into undeveloped territory, and improved their property as the territory developed and was justified by the traffic. The results have proved their wisdom abundantly.

Commercial Importance, Depth, Etc.—The depth of the channel, or rather the 6-ft. draft of vessels, was fixed by Congress, and the engineers connected with the work are in no way responsible for it. It is certainly ample, however, for ordinary flat-bottom river steamboats, and for ordinary coal barges of 500 tons' burden.

Unlimited arguments can be made for or against the commercial importance of these improvements. That is a matter to be considered more especially by Congress than by the engineers on the work, except in so far as they are called upon by Congress for information. In the opinion of the writer the commercial value of the work will have to be determined finally by experience. If it regulates railroad freight rates in the region tributary to the improved rivers, and develops a large export coal traffic from Mobile, the public will be amply repaid for every dollar expended.

MEMOIRS OF DECEASED MEMBERS.

GEORGE SEARS GREENE,* Past-President and Hon. M. Am. Soc. C. E.

DIED JANUARY 28TH, 1899.

George Sears Greene, son of Caleb and Sarah Robinson Greene, was born at Apponaug, in the Town of Warwick, Rhode Island, on May 6th, 1801.

He was a descendant, in the seventh generation, from John Greene, a surgeon, who came to America in 1635, from Salisbury, England, and settled in Warwick in 1642. Among his descendants, in the fifth generation, was General Nathanael Greene, who was a distinguished officer in the American Army in the War for Independence.

The subject of this memoir entered the United States Military Academy at West Point in 1819. During the last year of his academic course he was Acting Assistant Professor of Mathematics. In June, 1823, he was graduated second in his class, was appointed Second Lieutenant of Artillery, and was detailed as Assistant Professor of Mathematics and Engineering in the Academy, and continued there until 1827, with the exception of four months' service in the Artillery School at Fortress Monroe in 1824. On leaving West Point he served with his regiment, the Third Artillery, on garrison duty in Maine, Rhode Island and Massachusetts, and on ordnance duty.

In May, 1829, he was promoted to a First Lieutenancy. On June 30th, 1836, he resigned his commission in the Army and entered on the practice of civil engineering, engaging in the construction of railroads in North Carolina, Maine, Massachusetts, Rhode Island and Maryland.

In 1856 he was retained by the Croton Aqueduct Department of New York City, and for six years was occupied in the design and construction of works for the extension of the water supply, comprising the large distributing reservoir in Central Park, 38 ft. deep, and covering 96 acres; the construction of a wrought-iron pipe, 90.5 ins. in diameter and 1 400 ft. long, on High Bridge, across the Harlem River; and the laying of a cast-iron pipe, 60 ins. in diameter and 4 116 ft. long, across Manhattan Valley.

There were at that time no examples of similar works of equal magnitude with these. The plans and specifications prepared for these works by Captain Greene were carefully elaborated, and

* Memoir prepared by J. James R. Croes, L. L. Buck and G. S. Greene, Jr., Members, Am. Soc. C. E., Committee appointed by the Board of Direction.

embodied several novel features in reservoir and pipe construction, among which were the precautions taken against leakage and loss of water by founding the puddle walls in the surrounding embankments on concrete laid in trenches in the solid rock; by constructing a division embankment, with its top 3 ft. below the full water line of the reservoir, and having in its center a puddle wall founded on concrete resting on rock, and having upon it a toothing wall of brick masonry, 4 ft. high, which the puddle embraced; by lining the water faces of all embankments with a pavement of masonry on a bed of concrete; and by requiring all embankments to be built up in thin layers rolled with a grooved roller.

Another novel practice was the use of concrete in large masses (rendered practicable by the invention of the stone crusher in 1858 by Eli Whitney Blake), and the reduction of cost and increase of specific gravity of the mass by the insertion of large unwrought stones. There was also no precedent for the construction of the 90-in. wrought-iron pipe across High Bridge, more than ¼ mile long supported on saddles which rest on rollers, thus permitting changes in length caused by variations in temperature to be taken up by a stuffing box at each end of the pipe.

In the case of the pipe across Manhattan Valley, which was of 12 ins. greater diameter than any cast-iron water pipe previously manufactured, Captain Greene's investigations, of the material and methods of manufacture then existing, led him to discard the ordinary hub joint and to have the pipes cast as straight cylinders, with the joints made by half-sleeves bolted together. No leaks or fractures have occurred in these pipes, while, in the case of a line of 6-ft. cast-iron pipe with hub joints, laid some years later, under other auspices, most of the joints failed under pressure when water was turned on.

The works which Captain Greene had designed were well under way when the attempt was made to destroy the integrity of the Union. He responded promptly to his country's call, and went to the front in January, 1862, as Colonel of the 60th Regiment, New York Volunteers. On April 8th, 1862, he was commissioned as Brigadier General, United States Volunteers, and served in the Army of Northern Virginia and the Army of the Potomac until September, 1863, participating in the battles of Cedar Mountain, Antietam, Chancellorsville, numerous minor engagements, and Gettysburg, where, on the night of July 2d, 1863, his brigade, reduced to less than 1 500 men, held the extreme right of the Federal line on Culp's Hill for four hours against repeated assaults of the enemy, and defeated the strenuous efforts made to turn the Federal flank. In October, 1863, his brigade accompanied the Twelfth Army Corps to Tennessee, where he participated in several engagements, until October 28th, when, at the battle of Wauhatchie, a Minie rifle bullet passed through his mouth, coming

out through his right cheek. Even this severe wound only disabled him temporarily, thanks to his vigorous constitution and determined will. He did not even go into hospital, and in forty days was able to perform court-martial duty, to which he was assigned and on which he served until January 25th, 1865, when he rejoined his brigade in the field in North Carolina and took part in numerous minor engagements. On March 13th, 1865, he was commissioned as Brevet Major General, United States Volunteers, for "gallant and meritorious services." He served on garrison and court-martial duty in Washington, D. C., until April 30th, 1866, when he was mustered out of the service. This terminated General Greene's military career, except that in 1894 Congress passed a special act restoring him to the Regular Army, with the rank of First Lieutenant, and retiring him.

In civil engineering, however, it was only the beginning of another twenty years of active practice, at an age when most members of the profession are considering retirement from practice.

The Croton Aqueduct Department of New York City had just reached the determination to construct additional storage reservoirs in the Croton Valley, and, after a year spent in careful examination and comparison of several sites, the availability of which had been made apparent by a comprehensive survey of the entire valley made several years before, had decided, on March 17th, 1866, to construct the first of such reservoirs at Boyd's Corners, on the West Branch of the Croton River, where it was found that a dam, 78 ft. high from bed-rock and 670 ft. long at top, could be built to impound 2 700 million gallons. Of the design and construction of this work, General Greene took charge on May 1st, 1866. There were no American precedents for the construction of dams of this magnitude, and there was no literature on the subject in the English language. Indeed, the only published discussion of the principles which should prevail in the design of high masonry dams was the treatise of Sazilly in the *Annales des Ponts et Chaussées*, in 1853. But, with General Greene's thorough scientific education, his long experience with materials and modes of construction, and his habits of prompt decision, accentuated, no doubt, by his recent active participation in military affairs, it did not take him long to decide upon a plan of dam in which the materials most available should be utilized, with the least expenditure of time and money, and so disposed as to produce the most effective results; and on August 28th the contract was let for the construction of the Boyd's Corners Dam.

On May 11th, 1867, on the retirement of the late Alfred W. Craven, Past-President, Am. Soc. C. E., from the position of Commissioner and Chief Engineer of the Croton Aqueduct Department, General Greene was chosen as his successor, and held that position until the Department was abolished by the charter of 1870, and its duties

transferred to the Department of Public Works, on May 1st, of that year. General Greene remained in the service of the Department until January 11th, 1871, as an assistant engineer.

From 1868 to 1871 he was also Consulting Engineer to the Morrisánia Survey Commission which inaugurated the system of exact topographical surveys and monumenting of base lines and of street lines which has since been extended over the whole of what is now termed the Borough of the Bronx, under the specifications framed by the engineers to this Commission. On August 24th, 1871, he was appointed Chief Engineer of the newly organized Board of Public Works of the District of Columbia, with instructions to devise a system of sewerage for the City of Washington. He made a report on this subject, but his connection with the works terminated on June 24th, 1872, and he returned to New York, and, from October, 1872, to September, 1873, was Consulting Engineer, to the Department of Public Parks, on communications across the Harlem River by bridges and tunnels.

In the year 1873 he was also occupied as one of a Board of Engineers to examine the surveys and estimate the cost of a ship canal from Lake Champlain to the St. Lawrence River; and as a member of an Engineer Commission to test the working and construction of water meters for New York City. He was also called upon to examine projects for extending and improving the water supply of Detroit, and on August 15th, 1874, in conjunction with his colleague in the investigation, General Godfrey Weitzel, reported that filtration was necessary to ensure purity in the water drawn from either Lake St. Clair or the Detroit River. This suggestion was too far in advance of the times to be regarded favorably by the authorities.

On June 1st, 1874, he was appointed Civil and Topographical Engineer to the Department of Public Parks of New York City, in charge of the designing of the street system for the then recently annexed district north of the Harlem River, now designated the Borough of the Bronx. Some studies of plans for streets were made by him, but on November 5th, 1875, he was transferred to the position of Engineer of Construction of Streets and Sewers, in that District, which office he held until October 3d, 1877. The most important of the works of which he had charge during this period was the main outfall sewer of Brook Avenue, which drains an area of 2 615 acres and has a gradient of only 1 ft. in 2 618 ft. It is an arched conduit of brick, 12 ft. wide and 9.88 ft. high, the bottom being below the water level in the Harlem River, into which it discharges. The specifications for this work were very complete and thorough, and were deemed worthy of preservation, as an example of what such a document ought to be, in the American reprint of Baldwin Latham's book on Sewerage, issued in 1877.

In the consideration of the question of rapid transit in New York City, between 1866 and 1880, General Greene took an important part. The Legislature of 1866 designated Mr. A. W. Craven, the engineer of the Croton Aqueduct Board, as one of the members of a Commission to report upon the most advantageous and proper routes for the transportation of passengers in New York City; and the task of examining critically the numerous suggestions made to that Commission devolved largely upon General Greene, Mr. Craven's associate in this work. The conclusion reached by the Commission was that the best method of attaining the desired end was by the construction of underground railways. The New York City Central Underground Railway Company was incorporated in 1869, and at once caused examinations of its proposed routes to be made by W. W. Evans, E. S. Chesbrough and General Greene, and their report, estimating the cost of the road from City Hall Park to Harlem River at $17 625 301, and expressing the opinion that "the work in question will pay," was presented on October 19th, 1869. Capital could not be procured to carry out these plans. In 1871 General Greene, as consulting engineer to the Beach Pneumatic Railway Company, advocated the construction of a railroad under Broadway. When underground roads were found to be impracticable for the time, he was engaged in 1873 to consider the plans for the construction of the Greenwich Street elevated railway from Rector Street to Thirtieth Street. Six years later, when the extension of the elevated railroad system was agitated, he was appointed engineer to the Rapid Transit Commission of April 2d, 1879, which established the route and prepared the plans for the crossing of the Harlem River at Eighth Avenue and 155th Street, and organized the West Side and Yonkers Rapid Transit Company, which built the bridge there. This Commission also organized companies and had plans and specifications prepared for a rapid transit railroad from the City Hall to Forty-second Street, and for several new lines of road in the Borough of the Bronx, but the Courts decided that their powers had been exhausted by the organization of one company.

Of the hydraulic and sanitary works on which, at about this period, General Greene's opinion was sought, may be mentioned the water supply for Yonkers, New York, in 1874; projects for the sewerage of Providence, Rhode Island, in 1875; and the enlargement of the water-works of Troy, New York, in 1877. In 1883 he was one of the engineers called upon by the Commissioner of Public Works of New York City to advise upon the plans prepared for the New Croton Aqueduct.

On March 6th, 1886, he was appointed by the Aqueduct Commissioners as one of a Board of Examining Engineers to investigate charges which had been made affecting the management and the con-

dition of the work which had been done. Such was his physical vigor, even at his advanced age, that he insisted on walking through the entire length of the tunnels, examining closely everything as he went, a task to which his associates, Generals Newton and Gillmore, found themselves unequal, although nearly twenty-five years his juniors. This was his last professional service, but he maintained his mental vigor and much of his physical strength until his death, which occurred on January 28th, 1899, at his home in Morristown, New Jersey.

In his whole career as a civil engineer, General Greene, while cautious and conservative, was progressive, and kept fully abreast of all advances in his profession. He was one of the original members of the American Society of Civil Engineers, established November 5th, 1852, was President of the Society for two years beginning November 3d, 1875, and was elected an Honorary Member on October 26th, 1888.

In social life he was fond of companionship, and was greatly beloved by a large circle of friends of all ages to whom the stores of information acquired in his long career of close study and acute observation, and preserved in his remarkably retentive memory, were freely unfolded. An enthusiastic member of the Military Order of The Loyal Legion, the reunions of that association were a source of great enjoyment to him as well as to his old comrades. He esteemed particularly the hereditary feature of the organization, taking, as he did, great interest in genealogical studies. He was, for several years, President of the New York Genealogical and Biographical Society. He was, also, for thirty-one years, a member of the Century Association of New York City.

On July 14th, 1828, he was married to Elizabeth, daughter of David and Mary (Atwell) Vinton, of Providence, Rhode Island. She died on December 26th, 1832. On February 21st, 1837, he married Martha, daughter of Samuel Dana, of Charlestown, Massachusetts, for many years a Representative in Congress. Five sons and a daughter were the offspring of this marriage.

LORENZO RUSSELL CLAPP, M. Am. Soc. C. E.*

Died August 13th, 1902.

Lorenzo Russell Clapp was born in Quincy, Massachusetts, on April 25th, 1843. He was descended from Thomas Clapp, who settled in Scituate in 1640, and on his maternal side, was great-grandson of Captain Jonathan Clark.

In 1850 Mr. Clapp's family moved from Quincy to Dorchester. He was graduated from the Dorchester High School in 1860, and in 1861 he enlisted in the United States Navy, where he served until the close of the Civil War. He studied civil engineering under General Minot, later Chief Engineer of the City of Boston. At the same time he served a three-years' apprenticeship in the office of the late Thomas Doane, M. Am. Soc. C. E. His first experience in the field was with the Old Colony and Cape Cod Railway.

In 1869 he came to Brooklyn, New York, and received the appointment of Assistant Engineer in the Department of Public Works. He served in this department under the following Chief Engineers: The late Moses Lane, M. Am. Soc. C. E.; the late Julius W. Adams, Past-President, Am. Soc. C. E., and Robert Van Buren, M. Am. Soc. C. E. He was Resident Engineer in charge of building the Hempstead Storage Reservoir; he also had charge of several important improvements on the Brooklyn water-shed.

In 1882 he was placed in charge of the Brooklyn Relief Sewer system, and was Constructing Engineer for the great tunnel sewers in North Second Street and Greene Avenue, in that city.

Besides work for the City of Brooklyn, Mr. Clapp was associated for several years with the late Austin Corbin, in the Long Island Improvement Company's Fort Pond Bay surveys, Manhattan Beach improvements, etc.

Mr. Clapp was considered an expert in tunneling and sewer construction; as an engineer in construction he was well known to those of his profession, and he won the respect and confidence of all who were associated with him.

In 1894, after Mr. Van Buren left the Brooklyn Department of Public Works, he also resigned and entered the contracting business. This work, however, was uncongenial to him, and worry over several bad contracts, doubtless, had much to do with his breaking down and sudden death.

Since 1870 Mr. Clapp's home has been at Hempstead, Long Island. While resident there, in charge of the Storage Reservoir, he married,

* Memoir prepared by Robert Van Buren, M. Am. Soc. C. E.

in 1874, Miss Suzannah J. Smith, of Foster's Meadow. His wife and one son survive him.

Mr. Clapp was devoted to his profession, and found, in its demand for accuracy and truth, an atmosphere well suited to the conscientiousness and clear, cool judgment that was his natural heritage. He became a Member of the American Society of Civil Engineers on February 1st, 1888.

SAMUEL BARRETT CUSHING, M. Am. Soc. C. E.*

DIED DECEMBER 2D, 1888.

Samuel Barrett Cushing, Junior, was born in Providence, Rhode Island, on July 21st, 1846, and was the son of a Member of the American Society of Civil Engineers who was considered among the first of the hydraulic engineers of his day.† Young Cushing's early education was obtained in the public schools of Providence; and in May, 1866, on being graduated from the High School, he entered his father's office. On the death of his father, in 1873, Mr. Cushing succeeded to the business. His ability was recognized by the Supreme Court of the State, which appointed him to succeed his father, and gave him authority to regulate the taking and use for power of the water of the Blackstone River at Pawtucket and Woonsocket, under the decree based upon his father's investigations and report.

The rearrangement of the railroad terminal facilities in Providence, recently accomplished, a matter in which the municipality and the railway corporations were jointly concerned, was for many years a subject for consideration and report by various commissions. The elder Cushing was engineer to one of the earlier of these bodies, the Commissioners of the Cove Lands, which presented its report in 1873, but, as he was in feeble health when the engineering work was done, the plans submitted by this commission were those of the son. Another work of consequence in the elder Cushing's charge at the time of his death was the building of the dam of the Flat River Reservoir, which flowed an area of 900 acres, and this work the young man carried to completion.

In railroad work, Mr. Cushing made preliminary surveys for a line from Ashland to Lowell, Massachusetts, passing through Framingham, Wayland and Concord, also a line from Northbridge to Milford, Massachusetts, and made the preliminary survey and located the Webster Branch of the Boston and Albany Railroad, from Webster to Rochdale, Massachusetts. He was the local engineer at Providence of the Boston and Providence and the Providence and Worcester Railroads, and constructed the East Providence Branch of the latter road, from Valley Falls to tidewater in East Providence.

Much of his practice was devoted to foundations. He built the Arkwright Dam, the Blackstone Dam, the Barden Reservoir Dam and the Happy Hollow Dam, and was Consulting Engineer on the construction of the dam of the Diamond Hill Reservoir for the Pawtucket

* Memoir prepared by the Secretary from information furnished by Edwin P. Dawley and George C. Tingley, Members, Am. Soc. C. E., and W. C. Simmons, Esq.

† For Memoir of Samuel Barrett Cushing, Senior, see *Proceedings*, Am. Soc. C. E., Vol. i, p. 43.

Water-Works. On the construction of these works, he was appointed by the Court, as Commissioner, to Award Damages to the Mill Owners of Pawtucket and Central Falls for water taken from the Abbott Run by the water-works.

He was considered by the Town of Cumberland, Rhode Island, as its engineer, and designed and built several highway bridges over the Blackstone in that town. Six bridges over the Abbott Run, a tributary of the Blackstone, were also built by him. Among the larger single pieces of work he undertook were the coal pockets on the Lonsdale Company's wharf at Providence, and the foundations for the large chimney of the Narragansett Electric Lighting Company in Providence,* on which he was engaged at the time of his death.

He was of quiet and gentlemanly tastes and was highly esteemed by those with whom he was associated.

Some of the Members of the American Society of Civil Engineers commenced their careers in the office of the Messrs. Cushing, among them being Messrs. Elmer L. Corthell, Desmond FitzGerald, J. Albert Monroe, George C. Tingley, Richard H. Tingley, Henry W. Parkhurst and William H. G. Temple.

Mr. Cushing was elected a Member of the American Society of Civil Engineers on June 1st, 1887.

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* *Transactions*, Am. Soc. C. E., Vol. xxv, p. 1.

CHARLES FLETCHER HILLMAN, M. Am. Soc. C. E.*

DIED JUNE 14TH, 1902.

Charles Fletcher Hillman was born on December 19th, 1835, at Albany, New York, and was educated at the New York State Normal School.

From November, 1853, to February, 1855, he was located at Windsor, Canada West, as Assistant Draftsman to Associate Chief Engineer William Scott, of the Great Western Railway of Canada, and was employed on surveys of the proposed branch from Amherstburg to Baptiste Creek. In 1855 he was Assistant Engineer on the preliminary plans and estimates for the Canada Southern Railway, from Niagara Falls to Detroit, Michigan; and Principal Assistant Engineer on surveys for a proposed harbor at Two Creeks, on the North Shore of Lake Erie. From 1855 to 1857 he was Assistant Engineer on the location and construction of the Detroit and Milwaukee Railway, between Ionia and Grand Haven, Michigan.

In 1857 he accepted an engagement from Mr. Henry Meiggs to work on the "Ferrocarril del Sur," in Chile, and from that year until 1864 he was Assistant Engineer to the late Anthony Walton Whyte Evans, M. Am. Soc. C. E., on the various State Railways of Chile, being engaged on the surveys and construction of the lines between Santiago and San Fernando, and between Santiago and Valparaiso.

Mr. Hillman was the Contracting Engineer for the Tongoy Railway of Chile, in the years 1865 to 1868; and from 1870 to 1876 he was Chief Engineer on the construction of the Palmilla Branch and of the completed portion of the Chile Southern Railway, from Santiago to Curico. In 1871 he undertook the construction of part of the tramcar lines in Valparaiso.

The last railways built by Mr. Hillman, before retiring from active service, were those between Angol and Traiguén and between Renaico and Victoria in 1884. In the construction of these roads he was in partnership with Mr. Stephen Mayers. On account of delays and difficulties caused by the construction of the high bridge over the Malleco, near Collipulli, which was under the charge of the Government Engineer, Mr. Lastarria, the latter section was turned over to the Government before it was fully completed.

On several occasions Mr. Hillman was appointed by the Chilean Government to give information on technical matters, and was always a warm friend of the Chileans, both at home and abroad.

* Memoir prepared by the Secretary from papers on file at the Society House.

Some years ago Mr. Hillman introduced the manufacture of matches in Chile by the establishment of a factory at Rancagua. This factory was afterward removed to Santiago. During the last years of his life he acted as one of the Directors of the Santiago Gas Company, and to his influence are due the facilities given by the Company for the use of gas in the industries.

Mr. Hillman was one of the most prominent and influential members of the American colony in Santiago. He was a member of the Board of Trustees of Union Church, of the Sunday School, and of various philanthropic and social institutions. On several occasions he delivered lectures on behalf of charitable objects. He contributed to the press a number of articles on important railway problems, several of them being on the relative merits of English and American systems of equipment and rolling stock. Under the pseudonym of "Quien Sabe," he published, in 1901, a volume of some 300 pages, entitled "Old Timers, British and American, in Chile," which is a most valuable contribution to the history of Anglo-American influence in the growth and development of Chile. This work first appeared as a series of articles contributed to *Our Young People*, of which Mr. Hillman was, for several years, one of the Associate Editors.

Mr. Hillman took great interest in the educational work of his church, and not only contributed toward the outfit of the college established in Santiago, but used his influence with others for the same purpose. He took special interest in becoming acquainted with the scholars, individually, and accepted honorary membership in the Alumni Association.

In 1862 Mr. Hillman was married to Miss Caroline Haviland, a daughter of Samuel Frost Haviland, Esq., an American gentleman, resident in Chile. Of the seven children born to them only three remain.

In 1896 and 1897, accompanied by his family, Mr. Hillman made an extended visit to the United States, his first visit since his arrival in Chile. They came by way of San Francisco, and returned by the East, visiting Europe on the way back. During 1901, in company with his wife, he made another trip to the United States to visit his youngest daughter who was here in school. He did not remain long, however, on account of the illness of Mrs. Hillman.

Mr. Hillman was elected a Member of the American Society of Civil Engineers on July 5th, 1876. He was elected a Member of the Institution of Civil Engineers on April 4th, 1882. He was one of the founders of the "Sociedad de Fomento Fabril," was a member of the Council of Directors, and was zealous in the work of that society.

ARTHUR STANLEY HOBBY, M. Am. Soc. C. E.*

Died May 28th, 1902.

Arthur Stanley Hobby was born in Manchester, England, on March 15th, 1836, being the eldest of three sons of Benjamin Hobby, a manufacturer of wooden silk-working machinery. He was educated in St. Saviours School, at Manchester, and was articled as a "pupil teacher" in that school as the result of competitive examinations, according to the methods in vogue at that time. Upon finishing his term in 1854, he entered the employ of a large shipping firm, Jackson and Company, of Manchester, where he remained for three years. Not finding commercial business congenial, he articled himself in 1857 for a term of seven years to Mr. John Shaw, Civil and Mining Engineer, of Derby, England, who, in addition to a large general practice, was engineer to large coal-mining interests in that district. Due to his sound judgment in engineering matters, Mr. Hobby soon became an important assistant to his employer, and was entrusted with many problems of importance. In 1864 he was engaged by Mr. J. H. Arkwright to make surveys of an extensive estate in Herefordshire, upon the completion of which he established a private practice at Kingston, where he also built and operated on his own account a steam brickyard, using the patent Hoffman kilns. It is believed that this was quite a departure from the regular methods of brick manufacture at that time, but it was quite successful. While at Kingston Mr. Hobby married Miss Maria Elizabeth Bishop.

In 1871, due to the failing health of his wife, he moved to Virginia and tried farming, but soon drifted into his professional work again, holding positions on various projects in Virginia and Kentucky, among them being the Chesapeake and Ohio Railroad, the proposed Big Sandy Railroad, and various surveys of coal properties in Kentucky and Ohio.

In 1876 he settled in Cincinnati, Ohio, and became City Sewerage Engineer, under Latham Anderson, M. Am. Soc. C. E., City Engineer, both having been elected to office on the great reform wave which swept the city that year. Until that time Cincinnati had been drained by a very imperfect system of "combined" sewers, and Mr. Hobby introduced what is now known as the "separate" system, but was then known throughout that locality as the "Hobby" system. It will thus be seen that Mr. Hobby was one of the original workers along that line, and Cincinnati was one of the first cities in the country to adopt that system, notwithstanding the fact that the public

* Memoir prepared by his son, Arthur Stanley Hobby, Jr., Assoc. M. Am. Soc. C. E.

offered great opposition and ridiculed the system severely. The idea that a city of the size of Cincinnati could be drained by small pipe sewers was preposterous! He, however, had the courage of his convictions, and his arguments were so sound that he won the day, and his plans were adopted. During the four years that he remained in office a great deal was done to put the city on a proper sanitary basis, and the plans and regulations prepared by him are still followed, with but little if any variation.

About the year 1880 another political turnover put both Colonel Anderson and Mr. Hobby out of office, and the firm of Anderson and Hobby was formed, which enjoyed a very large practice.

In 1885, or the early part of 1886, owing to the failing health of Mr. Anderson, the firm was dissolved, and Mr. Hobby continued the business in his own name. Among the most notable of Mr. Hobby's works during that period were the design and construction of the plant and Town of Ivorydale, for the Proctor and Gamble Company, the design and construction of the plant and Town of Addyston for the Addyston Pipe and Steel Company, these plants at the time being the largest of their kind in the United States. He also introduced his sewer system into a number of towns and villages of Southwestern Ohio. His advice on sanitary matters was sought by many towns throughout the Central West. He was for many years Engineer to the Villages of Clifton and Avondale, Ohio, now a part of Cincinnati, and throughout that section of the country many important works stand as monuments to his sound judgment and skill.

In 1882, at Woodsdale, a small town near Hamilton, Ohio, he constructed a wooden dam across the Big Miami River, where the banks and bed of the stream were composed of shifting sands, and where three dams had already failed. The purpose of the dam was to maintain a valuable water-power right. The dam was built during the winter, under great difficulties, and was inundated by a heavy freshet when about half built, but it is still there.

In 1898 he gave up his practice in Cincinnati, and went for a short time to Philadelphia and then to Southern California. He died in Los Angeles on May 28th, 1902, after a stroke of paralysis followed by a lingering attack of typhoid fever, and leaves a widow, three daughters and a son.

Mr. Hobby was elected a Member of the American Society of Civil Engineers on June 6th, 1894.

MILTON GROSVENOR HOWE, M. Am. Soc. C. E.*

DIED JUNE 19TH, 1902.

Milton Grosvenor Howe was born at Methuen, Massachusetts, on August 16th, 1834.

He was graduated from Dartmouth College in 1854, having taken both the academic and scientific courses. His first work was on a survey for a road which was projected to run north from Saratoga, New York, through the Adirondacks to Sackett's Harbor. The surveying party, in charge of the late John Newell, M. Am. Soc. C. E., spent a winter in the mountains, but the road was never built.

In 1855 Mr. Howe went to Illinois, and was employed on the Illinois Central Railroad until the panic of 1857 caused the stoppage of all construction. His next work was in Iowa, but the stringency in the money market made business dull, and he found himself at the beginning of winter without occupation. Having a fondness for study, he decided to put in the time reading law, and so rapidly did he acquire knowledge that in the spring he passed the examinations and was given a license to practice. He was devoted to engineering, however, and, believing that greater opportunities existed in the Southwest, besides desiring a warmer climate, he went to Texas in the fall of 1859, and entered the service of the Houston and Texas Central Railroad.

At the beginning of the war he enlisted in the Army of the Confederacy as a volunteer in Company A of the Twenty-sixth Texas Cavalry, De Bray's Regiment. Within a year he was transferred to the Engineering Corps and given the rank of Captain. He was an ardent exponent of the cause of the South, and it was by his skill as an engineer and inventor that two of the five cannon used so effectively in the battle of Sabine Pass were improvised.

At the close of the war Captain Howe again entered the service of the Houston and Texas Central Railroad, and held the positions of Chief Engineer, General Superintendent and Division Superintendent.

In 1885 he was made Receiver of the Houston, East and West Texas Railroad. After this road, under his able management, had passed from under the Receivership, he became Vice-President and General Manager. In 1897, on account of poor health, he retired from active work. In 1899 Captain Howe was appointed Chief Engineer of the Houston and Texas Central Railroad, but resigned within the year, owing to poor health.

Many of Captain Howe's best efforts were directed toward advanc-

* Memoir prepared by the Secretary from information furnished by Mrs. M. G. Howe.

ing the interests of the City of Houston, of which he was an alderman for six years.

In 1873 he was married to Miss Jessie W. Briscoe, who, together with one son, Milton Howe, of the Engineering Department of the Houston and Texas Central Railroad, survives him.

Mr. Howe was elected a Member of the American Society of Civil Engineers on October 16th, 1872. He was appointed a Director on October 21st, 1901, to fill the unexpired term of the late George A. Quinlan, but at that time was in such poor health that he had to decline the appointment.

GEORGE WASHINGTON HOWELL, M. Am. Soc. C. E.*

Died February 15th, 1901.

George Washington Howell was born at the Howell Homestead, Littleton, Morris County, New Jersey, on December 21st, 1835, his father being the Honorable Edward Howell, and his mother Mary Lee, both descendants of old colonial families. When a lad of sixteen, young Howell accepted the position of teacher in a small country school near his home, and four years later he entered the New Jersey State Normal School, where, after being graduated, he remained as instructor in mathematics and languages for several years. Finding that his talents lay in the engineering line, however, he severed his connection with the Normal School, and studied civil engineering.

For some time he was associated with the late Professor George H. Cook, State Geologist of New Jersey, preparing the topographical maps and geological reports of the State, and it may be interesting to know that he prepared and made the first four sheets of the New Jersey State Topographical Survey.

Mr. Howell was at one time Constructing Engineer of the New Jersey West Line Railroad, and of the Western New York and Pennsylvania Railroad, being also connected with the Delaware, Lackawanna and Western Railroad at the Hoboken Terminal, the Boonton Branch and the Binghamton Extension.

Mr. Howell was Engineer for the Morris Aqueduct, for Morristown, New Jersey, and designed many of the reservoirs. He was also Engineer for the State Hospital, at Morris Plains, for which he designed the water-works and sewage disposal systems, as well as the landscape work. He was Engineer for Morris County and Morristown, New Jersey, for many years.

In later years he made a special study of drainage, water supplies, etc., and was appointed a member of the original Commission for the Drainage of Flow Lands along the Passaic River, serving as Secretary and Engineer until his death. In 1879, in conjunction with J. James R. Croes, M. Am. Soc. C. E., Mr. Howell made an exhaustive report on an investigation of the possible source of water supply for Newark, New Jersey, and this investigation brought him into many of the water suits of recent years. He was also connected with the East Jersey Water Company in investigations for water supply and installing the water-works of Montclair, New Jersey.

In addition to his engineering work, which, in later years, was principally as expert and consulting engineer on water-works, sewage disposal, bridges, etc., Mr. Howell was connected with many organi-

* Memoir prepared by James Owen, M. Am. Soc. C. E., and from a Memoir published by the New Jersey State Sanitary Association.

zations not bearing directly upon his profession. He was a member of the State Board of Education, of the New Jersey State Sanitary Association, of the Board of Directors of the Morristown Memorial Hospital, for which he was also Secretary, of the Washington Association of New Jersey, and of the Sons of the American Revolution.

In all this busy life, however, Mr. Howell still found time for his church work, and was President of the Board of Trustees and Deacon for life of the Morristown Baptist Church. He was also Recording Secretary of the Morris County Sabbath School Association and Secretary of the North Jersey Baptist Association. He served also as Treasurer of the Young Men's Christian Association.

From early manhood Mr. Howell had a keen love for reading, and spent much time in his library, where he also wrote many contributions to the press, especially upon historical matters, as he was well versed in State and local genealogical matters of historical interest.

Truly revered and loved as Mr. Howell was by all his many associates on the boards and in the associations of which he was a member, yet it remained for those who knew him in his private life to value him at his true and sterling worth. He was married on December 31st, 1862, to Rachel M. Cornish, of Gillette, New Jersey, and had five children, all of whom survive him.

Throughout his entire life Mr. Howell was the model of a Christian gentleman, thorough and conscientious in his work, honorable in business dealings, and singularly courteous and winning in his manners toward all with whom he came in contact.

Mr. Howell was elected a Member of the American Society of Civil Engineers on May 2d, 1888.

MORITZ LASSIG, M. Am. Soc. C. E.*

DIED JANUARY 7TH, 1902.

Moritz Lassig was born in Rochlitz, Saxony, on January 23d, 1831. He attended the Architectural School in Chemnitz, and, having completed his studies there, came to America, where he arrived in the spring of 1851. In the fall of the same year he settled in Chicago, having accepted a position as Engineer and Superintendent with the firm of L. B. Boomer and Company, later the American Bridge Company, of Chicago. This position he held until 1853. From 1871 to 1881 he was engaged in designing and constructing bridges throughout the West.

In the year 1876 he established his own shop on South Clark Street, near Sixteenth Street. The present shop, at Clybourn and Wrightwood Avenues, at which he achieved his principal success as a bridge builder, was founded about 1885. In 1881 he formed a partnership with John P. Alden, under the names Lassig and Alden Bridge and Iron Works of Chicago, and Alden and Lassig Bridge and Iron Works of Rochester, New York. Mr. Lassig was active as the President and guiding spirit of the Lassig Bridge and Iron Works until the fall of 1900, when he sold his interests to the American Bridge Company and severed his connection with the firm.

He was on the eve of departure for Europe, having made all arrangements to leave Chicago on January 8th, but was overtaken suddenly by death. His remains have been interred at Heidelberg, Germany.

His widow, Marie Lassig, and two married daughters, Mrs. Emma Bartholomay and Mrs. Ida Olinger, survive him.

Moritz Lassig was elected a Member of the American Society of Civil Engineers on April 2d, 1884. He was also a Member of the Western Society of Engineers, of the Technical Club, Concordia Masonic Lodge, Germania Männerchor, and the Chicago Schuetzen Verein.

* Memoir prepared by the Secretary from information furnished by Mr. Henry Bartholomay, Jr., and from papers on file at the Society House.

THOMAS LAIDLAW RAYMOND, M. Am. Soc. C. E.*

Died November 15th, 1901.

Thomas Laidlaw Raymond was born in New Orleans, Louisiana, on May 22d, 1853, and received his preliminary education in the schools of that city. In 1870 he entered the University of Virginia, from which he was graduated in 1874, with the degrees of Civil and Mechanical Engineer and Bachelor of Science.

From January until October, 1875, he was a draftsman in the office of the City Surveyor of New Orleans. From October, 1875, until July, 1876, he taught mathematics in the Preot School, after which he became bookkeeper and cashier for the firm of Elkin and Company.

In May, 1879, Mr. Raymond entered the service of the Government, as U. S. Assistant Engineer, and, during the succeeding nine years, was stationed, successively, at South Pass, Sabine and Blue Buck Passes, Sanibel Island, Bayou Lafourche, Bayou Courtablau, and other points on the Gulf Coast.

From August, 1888, until December, 1896, he was connected with the Board of State Engineers of Louisiana, as Chief Draftsman. In December, 1896, he was chosen as Principal Assistant Engineer of the Drainage Commission of New Orleans, which position he filled until his death.

At the request of the Sewerage and Water Board of New Orleans, Mr. Raymond served as a member of their Advisory Council of Engineers. His interest in this work and in the public welfare was so great that, in view of a legal doubt concerning his right to payment, these services were rendered gratuitously. The duties of these several positions were discharged with an ability and fidelity which won for him the esteem of his employers and associates.

Mr. Raymond's success as an engineer was due partly to special fitness for such work, partly to close observation and application, and largely to a high standard of personal and professional duty. Combined with this equipment he had a good heart and a generous manner, which secured the warm regard of all who had the privilege of serving with him. His associates will all join in a recognition of his ability as an engineer and his value as a citizen and friend.

He was married in 1889, and leaves a widow, a son and two daughters. He was active in organizing the Louisiana Engineering Society, and served as its second President, in 1897. His interest in that society continued until his death.

Mr. Raymond was elected a Member of the American Society of Civil Engineers on October 6th, 1897.

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* Memoir prepared by the Secretary from papers on file at the House of the Society, and from a Memoir prepared by a Committee of the Louisiana Engineering Society and published in the *Journal* of the Association of Engineering Societies, Vol. xxviii, page 98.

NATHANIEL EDWARDS RUSSELL, M. Am. Soc. C. E.*

Died January 14th, 1902.

Nathaniel Edwards Russell, son of Charles P. and Louisa (Richardson) Russell, was born at Washington, D. C., on February 24th, 1848. His father was a clergyman, a personal friend of Abraham Lincoln, and delivered the sermon at the funeral of the murdered President.

Mr. Russell's education began in the schools of Washington, and he then took a practical course in mechanical engineering, serving his time in machine shops there.

In 1867 he entered the Rensselaer Polytechnic Institute, and was graduated in 1870, taking the degree of C. E.

His first position was that of Civil Assistant Engineer under General G. K. Warren, Corps of Engineers, U. S. A., his work being upon surveys and improvement of rivers and harbors in New England. Later, he was transferred to the Northwest, in the same service, and, as Assistant Engineer, was in charge of the work of establishing slack-water navigation on the Fox River. He also made numerous surveys. From May to November, 1871, he was in charge of the construction of a pile pier in the harbor at Menominee, Michigan. From November, 1871, to May, 1872, he was in the United States Engineers' office at Chicago, Illinois. From May to November, 1872, he was Assistant United States Engineer in charge of the lower section of the Wisconsin River improvement, and from the latter date until July, 1875, he was in charge of the detailed surveys and the construction of the upper and lower sections of the Fox River improvement.

From September, 1875, to February, 1876, Mr. Russell was Assistant Engineer on the construction of the Albany and Greenbush Bridge. From July, 1876, to January, 1882, he was with the Wiley and Russell Manufacturing Company of Greenfield, Massachusetts, designing and superintending the enlargement of their plant.

Mr. Russell was General Manager of the Alleghany Coal and Iron Company, of Richmond, Virginia, from January, 1882, to March, 1883, and President of that company from the latter date until July, 1885, and at the same time General Manager of the Henrico Coal Company, of Richmond.

He practiced afterward as a consulting engineer, with headquarters at Lansingburg, New York, designing and erecting bridges and roofs, among the latter being that of Harmanus Bleeker Hall, at Albany, New York. He was also in charge of the construction of the bridge

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* Memoir prepared by A. J. Swift, C. E.

connecting the tracks of the Utica, Clinton and Binghamton and the Utica and Black River Railroads, at Utica, New York.

For a short time he was Superintendent of the works of the American Graphophone Company.

From 1891 to 1894 he was General Manager of the Walter A. Wood Manufacturing Company, at Hoosick Falls, Massachusetts, and, later, resumed his practice as consulting engineer.

On July 1st, 1874, Mr. Russell married Miss Lucy Coleman Flack, daughter of Mr. David H. Flack, of Lansingburg, New York.

His death occurred at Lansingburg, New York, on January 14th, 1902, and resulted from a fall in alighting from an electric car in the summer of 1891, his senses of smell and taste remaining much impaired after his recovery from a severe illness following this accident. He died after a few moments of intense suffering, followed by several hours of unconsciousness. He leaves a widow and one daughter.

It is difficult to express in words the affection and esteem which were felt for Mr. Russell by every one privileged to know him. To him his friends were truly "knit with hooks of steel," and the announcement of his death made the world seem smaller to each and all of them.

He felt no wish to become conspicuous, professionally or otherwise, but shrank from notice, rather than courted it. He always undervalued his own ability, and worked zealously, intelligently and successfully, from love of the profession which he adorned. No thought or action below the highest standard of honor, kindliness, and generosity was possible to him, and every individual who knew him will endorse the absolute truth of these words. He deserved, if ever man did, to rank as a type of the dignified Christian gentleman, able and experienced engineer, man of catholic and cultivated taste and reading, devoted husband and father, and staunch friend; and, as a rare combination of all these qualities, his sorrowing friends will always remember him. Every community in which he has lived has benefited by his life and is a loser by his death.

Mr. Russell was elected a Member of the American Society of Civil Engineers on October 3d, 1888.

IRA ALEXANDER SHALER, M. Am. Soc. C. E.*

Died June 29th, 1902.

Ira Alexander Shaler was born in Ridgefield, New Jersey, on September 19th, 1862. He was the son of General Alexander Shaler, who served in the Army of the Potomac with distinction. His mother, before marriage, was Miss Mary McMurray, of New York City. Mr. Shaler was a direct descendant, in the eighth generation, of Captain Thomas Shaler, who came to America from Stratford on Avon, England, in 1662, and, with twenty-seven other emigrants, founded the town of Haddam, in Connecticut, where most of his descendants lie buried.

At the age of eight young Shaler was sent to a private school in New York. Three years later he entered the Public Grammar School, No. 55, from which he was graduated in 1878. For the next two years he studied at the College of the City of New York, and in all these schools his record was excellent.

On September 16th, 1880, he entered Cornell University as a sophomore, taking for the first year the general course of Science and Letters. Feeling, however, that this led to no particular profession, and desiring to work for some specific purpose, he decided to take the course of Civil Engineering. He was graduated, with the degree of B. C. E., in 1884. Two years later he received his M. C. E. from Cornell University.

Soon after graduation he entered the Engineer Corps of the Aqueduct Commissioners of the City of New York, being first engaged in the office of the Chief Engineer, and, later, in the field, on the construction of the New Croton Aqueduct. As Assistant Engineer he had charge of a section of the Aqueduct Tunnel on Manhattan Island, of the 135th-Street Gate House, and of the pipe line from 135th to 125th Streets.

His tastes, however, were more in the line of contracting than of engineering, and, therefore, he left the service of the Aqueduct Commissioners in 1889 to engage in contracting. Among the works executed by him, either alone or in partnership with others, were the Cornell Hydraulic Laboratory; the Titicus Dam, forming a reservoir for New York City; the United States fortifications on Gull Island; the Cold Spring (N. Y.) Dam; the water-works for Gloversville, New York, and the changing of the Madison Avenue car tracks to a conduit electric system.

At the beginning of the Spanish War, Mr. Shaler offered his services to the Governor of New York to raise an engineer regiment to

* Memoir prepared by Edward Wegmann, M. Am. Soc. C. E.

serve during the war, but, learning that the Governor had no authority to furnish such a regiment, he accepted an invitation from Colonel Eugene Griffin to assist him in organizing the First Regiment, U. S. Volunteer Engineers, and was appointed Captain of Company F. He accompanied this Regiment to Puerto Rico, and returned with it as Major of the Second Battalion.

In January, 1899, Major Shaler was married to Miss Mary Duncan Leverich. After spending a year abroad with his wife, he returned to New York to engage again in contracting, and obtained a sub-contract for Section 4 of the New York Rapid Transit Road. This section, which is entirely in tunnel, extends under Fourth Avenue from 34th to 42d Streets. On this work Major Shaler met with a series of almost unparalleled misfortunes. On January 27th, 1902, his supply of dynamite exploded, killing five persons and doing great damage to the Murray Hill Hotel and other buildings. While Major Shaler was exonerated from all blame in this matter, he was responsible for the damage done. This explosion ruined him, financially.

On February 22d occurred the conflagration of the 71st Regiment Armory at 34th Street and Fourth Avenue, which burned a part of Major Shaler's plant and tools. On March 21st some ledges of rock slid into the tunnel, causing such damage to four houses opposite that they had to be purchased by the contractor to avoid suits for damages. This series of disasters culminated on June 17th, when Major Shaler was inspecting his tunnel with Chief Engineer William Barclay Parsons and Deputy Chief Engineer George S. Rice. A mass of rock became detached from the roof and struck Major Shaler on the back of his head and shoulders, causing injuries of which he died on June 29th, 1902.

Major Shaler had a force of character that inspired with confidence all who came in contact with him. His mind was well balanced. In prosperity, as in adversity, he remained the same. He had a cheerful disposition, was very energetic, a disciplinarian, and yet so just that he was liked by those who worked under him. By nature cautious, he would discard defective machinery long before it was worn out. He was very conscientious, and followed what he conceived to be his duty even though it involved a loss. At the outbreak of the Spanish War, though engaged to be married, nothing could deter him from performing what he thought to be his duty, viz., to offer his services to his country.

His manly character, genial disposition and many other sterling qualities won him a host of friends. His unselfish nature was well shown by the first words he uttered after being stricken down by the heavy piece of rock that broke his back and inflicted other serious injuries. These words were:

"I am afraid that my back is broken, for I have no feeling in my

body. Take me to the hospital, so that Mrs. Shaler won't see me until she has been warned. Break the news to her just as gently as you can, then bring her to me. Bring, also, my father and mother. Let the work on this section go right on."

Major Shaler was a member of the Alpha Delta Phi, Barnard and Cornell Clubs, and a member of the Loyal Legion (second class). He was elected a Junior of the American Society of Civil Engineers on July 4th, 1888, and a Member on June 5th, 1895.

ARTHUR TOWNE THOMAS, M. Am. Soc. C. E.*

DIED OCTOBER 10TH, 1900.

Arthur Towne Thomas was born in the Town of Stowe, Vermont, on August 30th, 1862. His parents were Abijah Towne and Clarissa Slayton Thomas, and he was the youngest of five children, of whom three were boys. His education was obtained at the district school, with one year in the high school in his native town. He left school at the age of 17, and went to Minnesota in 1880 to begin work as Chainman on railroad surveys. In March, 1881, he was Transitman on the St. Paul, Minneapolis and Manitoba Railway, with which road he remained until January, 1883, being engaged on surveys and as Assistant Engineer in charge of construction.

From February, 1884, until March, 1888, he was connected with the Minnesota and Northwestern Railway (now the Chicago and Great Western). Beginning as Transitman, he was advanced to the position of Division Engineer, having charge of the construction of numerous high trestles and iron bridges. From March, 1888, to March, 1889, he was in charge of railway terminals and dock works at Superior, Wisconsin. He was then appointed first City Engineer of Superior, which office he held until March, 1900, with the exception of four years, during which he was engaged as Chief Engineer of the Superior Water, Light and Power Company and the West Duluth Light and Water Company, in designing and constructing water-works, gas and electric light stations. Part of this time was also devoted to the study of mining engineering and its application to exploration work in the iron and gold fields of Northern Minnesota and Ontario.

In October, 1900, while he was Superintendent of the Chippewa Copper Mine, at Brule, Wisconsin, he went out prospecting, and, not returning when expected, search was made and his body found in the woods. Upon investigation, it was found that he had been shot, the bullet piercing the jugular vein and spinal cord. By means of the peculiar bullet, a deer hunter was discovered who acknowledged that he had shot Mr. Thomas accidentally and then abandoned the body heartlessly.

The conscientious attention which Mr. Thomas gave to his work, and his faithful performance of it, together with his earnest desire to accomplish the best results for those whose interests were intrusted to him, gained the respect and friendship of all who knew him.

Mr. Thomas was elected a Member of the American Society of Civil Engineers on June 6th, 1894.

* Memoir prepared by George L. Wilson, M. Am. Soc. C. E.

ASHLEY BEMIS TOWER, M. Am. Soc. C. E.*

Died July 8th, 1901.

Ashley Bemis Tower was born in the Town of Windsor, Massachusetts, on June 26th, 1847, being the youngest son of Stephen D. and Esther E. Tower, and one of a family of ten children.

He was of the eighth generation descended from John Tower, who in 1637, during the religious troubles of that time, came from Hingham, Norfolk County, England, and settled at Hingham, Massachusetts, helping to found that town. Mr. Tower's ancestors on his father's side fought in both King Philip's and the Revolutionary Wars.

Early in 1854 the family moved to Dalton, Massachusetts, where he received a common-school education, and, with his father, worked upon the farm. At the age of twenty-one, Mr. Tower went to Newburg, New York, and there learned the carpenter's trade with an older brother.

In 1871 Mr. Tower left Newburg for Holyoke, Massachusetts, to enter the engineering office of another brother, D. H. Tower, where for seven years he studied and labored, preparing himself for the profession which he afterward made his life work.

During the fall of 1878 the brothers became associated under the firm name of D. H. and A. B. Tower, to carry on a business as architects and civil engineers. They were pioneers in the practical design of paper mills, and soon reached an enviable position in that branch of the profession.

During the thirteen years that they were together, a great many of the paper, pulp and fiber mills built in the United States were designed by them. They also furnished plans for mills erected in Canada, Brazil, Germany, Japan and Australia, and designed several textile mills.

During the years 1884 and 1885, Mr. Tower visited Europe twice in order to study foreign methods of paper-mill construction.

In January, 1892, the partnership was dissolved, and Mr. Tower carried on the business himself until February, 1893, when Mr. George F. Hardy became associated with him under the firm name of A. B. Tower and Company. This arrangement was continued until March, 1896, when Mr. Hardy retired, and again Mr. Tower went on with the work alone.

In 1897, owing to increased business and the necessity for a more central location, the offices were moved from Holyoke to New York

* Memoir prepared by J. W. Tower, Assoc. M. Am. Soc. C. E.

City. During October of the same year, the firm became Tower and Wallace, in which Joseph H. Wallace, M. Am. Soc. C. E., was junior member. Mr. Wallace retired in February, 1901, and Mr. Tower remained alone until the time of his death, which occurred very suddenly, at his home in Montclair, New Jersey, on July 8th, 1901.

For a number of years Mr. Tower was Consulting Engineer for the American Sulphite Company, and in 1881 was elected City Engineer of Holyoke, Massachusetts, which position he held for three years.

He was a man of fine physique, standing more than 6 ft. in height, always courteous, modest and frank.

In 1875 he married Miss Pamelia J. Fritts, who survives him. Mr. Tower was elected a Member of the American Society of Civil Engineers on October 3d, 1894; he was also a Member of the American Society of Mechanical Engineers, and of the Canadian Society of Civil Engineers, and was well up in Masonry, being a Knights Templar, and a 32° Mason in the Scottish Rite.

CHRISTOPHER CHAMPLIN WAITE, M. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1896.

Christopher Champlin Waite was born at Maumee City, Lucas County, Ohio, on September 24th, 1843. He was a son of Morrison Remick Waite, late Chief Justice of the Supreme Court of the United States, and Amelia (Champlin) Waite. He inherited a sturdy strength of character which served him well in a long period of private and public usefulness.

He entered the Rensselaer Polytechnic Institute in 1860, and was graduated in 1864. His first engineering employment was on the Coxsackie Railroad, in 1865 and 1866. During the next two years he was engaged on the Croton Aqueduct for New York City. In 1868 he became Chief Engineer of the Columbus and Toledo Railroad, and Chief Engineer and Superintendent of the Cincinnati and Muskingum Valley Railroad. In 1889 he became President of the Columbus, Hocking Valley and Toledo Railroad, and, under his administration, in spite of strikes and financial embarrassments from other sources, a property which had met with many reverses was converted into a paying investment. He was thoroughly conversant with all the details of the operation of the road, and, while he exacted obedience to a rigid discipline, there was a bond of fellowship between the president and the subordinate.

Mr. Waite was married to Lillie C. Guthrie, daughter of Julius C. Guthrie, of Zanesville, Ohio, on October 22d, 1868, and is survived by his widow and two children, Harry and Ellison.

Mr. Waite was interested in the public affairs of the communities in which he lived, and was at one time Vice-President of the Cincinnati Chamber of Commerce.

He was prominent in charitable work, served as a Trustee of the Children's Hospital, and as President of the Board of Trustees of the Ohio State Epileptic Hospital, at Gallipolis, in the organization and management of which he was deeply interested.

He was a lover of art, and had at his home, in Cincinnati, a fine collection of statuary and paintings.

Mr. Waite was elected a Member of the American Society of Civil Engineers on March 3d, 1880.

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* Memoir prepared by the Secretary from information furnished by H. S. Waite, Esq., and from papers on file.

OSCAR F. WHITFORD, M. Am. Soc. C. E.*

Died May 21st, 1902.

Oscar F. Whitford, third child and second son of Earl Hartwell and Asenath (Palmer) Whitford, was born on July 15th, 1833, in the Town of Northumberland, Saratoga County, New York. He lived on his father's farm until he grew to manhood, attending district schools, Schuylerville Academy, and a preparatory school at Charlotteville, New York.

After being graduated from Union College, Schenectady, New York, in the classical course, in 1858—having taken a part of the engineering course as extra, under Professor Gillespie—he went to Mississippi, where he taught school and sold machinery until 1861, when he returned to Union College and took a post-graduate course in chemistry, receiving the degree of A.M.

In 1862 he was a volunteer in the United States Army for four months, to escort and protect emigrants crossing the Western Plains and Rocky Mountains.

After this service he engaged in gold mining enterprises in California and Idaho. He left this work to accept the Chair of Mathematics and Civil Engineering in the People's College at Havana, New York, now Montour Falls.

For a period of ten years, up to 1876, he was employed on the New York State Canals, in the Engineering Department, in charge of work, first on the Chenango Canal and afterward on the Erie Canal.

Leaving this service he engaged in lead mining in Missouri for two years, after which he was an engineer on the construction of the Southern Kansas Railroad for a year.

The following year, 1880, was taken up in testing cements and in the duties of general storekeeper for the Plattsmouth Bridge.

Silver and gold mining in Colorado and Mexico occupied his attention from 1881 to 1887. During the last two years of this period he was Superintendent of the Santa Barbara Mines at Chihuahua.

From 1888 to 1890 he was employed as Engineer for contractors on railroads for the Chilean Government. Returning to the United States he was engaged as Assistant Engineer on the Michigan Central Railroad, then as General Inspector in the Bureau of Engineering of the City of Buffalo, New York, up to 1898. From that year up to the time of his death he was occupied with various engineering enterprises.

He was a man of kind disposition and remarkably even temperament. He was loyal to his friends, kind and considerate to his subordinates.

* Memoir prepared by S. J. Fields, M. Am. Soc. C. E.

He was unmarried, and is survived by two brothers and a sister.

Mr. Whitford was elected a Member of the American Society of Civil Engineers on December 6th, 1871, and contributed to the *Transactions* a paper* entitled "Closing Breaks in Canals, under Difficulties."

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* *Transactions*, Am. Soc. C. E., Vol. ii, p. 161.

REGINALD McKEAN, Assoc. M. Am. Soc. C. E.*

Died October 15th, 1901.

Reginald McKean was of Irish parentage, and was born in Liverpool, England, on September 2d, 1867. He came to the United States in 1871.

He was educated in the public schools of St. Louis, Missouri, supplementing his training there by various courses with specialists in mathematics, chemistry, etc., from time to time, as circumstances permitted.

After a few years spent in office work, failing health admonished him that a more active life was a necessity, and, in 1884, he secured a minor position with the Mississippi River Commission, engaged in dredging near St. Louis. A short time afterward he secured employment in the engineering department of the St. Louis and San Francisco Railroad Company. From that time until the fall of 1896 he was engaged constantly in railroad work, occupying various positions of increasing responsibility, and gained a wide experience in both maintenance and construction work, his field of labor being principally in Missouri, Arkansas and Indian Territory, and with the St. Louis and San Francisco and the Missouri Pacific Companies.

He left the United States in December, 1896, and spent four years in the Transvaal, South Africa, where he was in the employ of the French Rand Gold Mining Company, Limited, as Assistant to the Chief Engineer, having charge of the construction of new work. He was held in highest esteem by the management of this property, and, when war came on, and the officials, who were mostly Englishmen, were compelled to leave the country, he was left in entire charge of the property, a position of great responsibility, anxiety and annoyance, and requiring much patience and tact. In the latter part of 1900 it became possible for a limited number of the officials to return to the mines in the Transvaal, and Mr. McKean was granted a much-needed leave of absence, until such time as it was possible for the mines to resume operations. He left there on December 1st, 1900, and arrived at his home in Cincinnati, Ohio, at Christmas.

Shortly after his return to the United States, there being no prospect of an early termination of the war in South Africa, he accepted a position as Office Engineer in the Construction Department of the St. Louis and San Francisco Railroad Company, with headquarters at Sherman, Texas.

On June 26th, 1901, Mr. McKean was married to Miss Mary Cecilia Denis, of Bond Hill, Ohio, and they at once took up their residence in Sherman.

* Memoir prepared by J. F. Hinckley, M. Am. Soc. C. E.

He died on October 15th, 1901, as a result of an attack of appendicitis.

A distinguishing feature of all Mr. McKean's work was a conscientious thoroughness and absolute fidelity to the best interests of his employers. In his social life, he was a most agreeable companion, kindly considerate of others, and of unruffled serenity under all conditions.

Mr. McKean was elected an Associate Member of the American Society of Civil Engineers on May 2d, 1894.

KENNETH OAKE PLUMMER REINHOLDT, Assoc. M. Am. Soc. C. E.

Died February 6th, 1902.

Kenneth Oake Plummer Reinholdt was born in New Castle, Pennsylvania, on September 27th, 1866, and was the only son of the late Dr. J. B. Reinholdt, of that city. His earlier education was received in the public schools of New Castle. Later, he attended Allegheny College, at Meadville, Pennsylvania, and from there he went to Rensselaer Polytechnic Institute, at Troy, New York, from which he was graduated in June, 1890.

In the following year he was employed in the laboratory of the steel plant of the Phœnix Iron Company in Phœnixville, Pennsylvania. In December, 1891, he accepted the position of Inspector of lumber and piles with the Engineer of Construction of the Central Railroad of New Jersey, and in March, 1892, he was given charge of the construction of the terminal pier of the Navesink Railroad, at Atlantic Highlands, where he located and completed the station buildings for the main dock and, also, all necessary sheds, platforms, trestles, etc.

He continued in the employ of the Central Railroad of New Jersey, as assistant to the Engineer of Construction, until September, 1895. During that time he made surveys and plans, and had charge of the elimination of grade crossings at Elizabeth, Cranford and Bayonne. He also located bridges at Freehold and Lorillard, New Jersey.

In September, 1895, he opened an office in New Castle, Pennsylvania, for private engineering practice, doing general city and country work. He designed a number of large buildings, and was recognized as a young man of talent in more lines than one. He had considerable ability as an artist, and had a fine collection of drawings.

In June, 1896, he accepted a position in Pittsburg, Pennsylvania, as draftsman with the Butler and Pittsburg Railroad, and, later, was made Assistant Engineer of the Bessemer and Lake Erie Railroad, a position which he held at the time of his death.

During the summer of 1901, while in Greenville, Pennsylvania, superintending the building of a new branch of the railroad, he contracted a severe cold, which developed into tuberculosis. His health failed rapidly, and in the early winter he and his wife left for Phœnix, Arizona, in hope that a change of climate would restore his health. It was too late, however, to overcome the disease which was undermining his constitution, and his strength waned day by day until the end.

Mr. Reinholdt was characterized by great faithfulness to all details of his duties, and was devoted to his profession. There was every indication of a brilliant career for him, and his sudden death caused poignant sorrow to his many friends. He was a young man of upright

character, and possessed a manner so kind that he was always surrounded by a circle of admiring friends. In June, 1900, Mr. Reinholdt was married to Miss Belle Paine, of Youngstown, Ohio, and she and an infant daughter survive to mourn their irreparable loss.

Mr. Reinholdt was elected a Junior of the American Society of Civil Engineers on February 6th, 1894, and an Associate Member on October 7th, 1896.

JOHN WOODBRIDGE DAVIS, Assoc. Am. Soc. C. E.*

DIED NOVEMBER 7TH, 1902.

John Woodbridge Davis was the son of Dr. Edwin Hamilton and Lucy Woodbridge Davis. His father, a physician of New York, was Professor of Materia Medica and Therapeutics in the New York Medical College, and was noted also for his investigations and discoveries in the ancient mounds of the Mississippi Valley. His book on the subject was the first published by the Smithsonian Institution. The family stock was a mingling of New England and Southern elements.

Mr. Davis was born in New York on August 19th, 1854. He received his earlier education at public and private schools in New York and Ohio.

From 1872 to 1874 he was engaged as transitman on the Baltimore and Ohio Railroad. As a student in the School of Mines, Columbia University, Mr. Davis published his "Formulæ for Calculating Railroad Excavation and Embankment" (1876), which was immediately adopted for use as a textbook in the School of Mines and in other Colleges. He had entered the University as a sophomore in 1875. At the same time he was exhibiting literary ability. In October of that year a prize offered for an essay—the contest being open to all students of Columbia—was won by Mr. Davis' essay on "Philosophy," published in *Acta Columbiana*, December, 1875. Previous to that time he had published poems and articles in New York and Ohio newspapers. Now, he became editor of *Acta Columbiana*—1876-1877—and contributed largely thereto. He was graduated with the degree of Civil Engineer in 1878, when he was Class Historian.

From 1879 to 1882 he was in Tennessee, where he surveyed 900 000 acres of land—the domain of the Grundy Mining Company—extending into eleven counties of Tennessee, in the Cumberland Mountains. This position demanded not only ability, but also courage and character, for the mountain squatters and woodsmen were dangerous people who resented curtailment of their self-made privileges. So conspicuous was the success of Mr. Davis in this undertaking that he was offered another civil engineering position under The Tennessee Coal, Iron and Railroad Company. This he declined, through a desire to make his home in New York.

He had secured the degree of Doctor of Philosophy from Columbia in 1880—chiefly by five articles published in *Van Nostrand's Engineering Magazine* in 1879. These were:

"A New Rule for Calculating the Contents of Land Surveys,"

"The Prismoidal Formula,"

* Memoir prepared by J. S. Sewell.

"A New Center of Gravity Formula of General Applicability,"

"A New and General Moment of Inertia Formula,"

"A New Graphical Method for Finding the Center of Gravity of a Polygon."

In 1881 he published, in the *School of Mines Quarterly*, "Inaccessible Distances in Surveying"—a paper written in collaboration with Professor Henry S. Munroe.

In 1882 Mr. Davis instituted the Woodbridge Preparatory School, and conducted it with success until 1897. As an educator he approved himself to the public. He had the power of elevating the moral as well as the intellectual standard of his pupils. His personality was winning and his influence great.

In 1883 he contributed a chapter to Davies' "Surveying." In 1884 he was appointed Examiner of Civil Engineers for New York City, in the Municipal Civil Service Examination Board, being the first to hold such a position in this city. Papers descriptive of the Civil Service Examinations appeared in *Engineering News* in 1895.

In 1884 Mr. Davis was one of the Committee of Ten, appointed from the Alumni of the Columbia School of Mines, for the purpose of examining the condition of the school and making beneficial suggestions. The report of the Committee embodied fifty-five recommendations in regard to changes in the curriculum and the standard of admission.

For several years Mr. Davis had spent time in research for the purpose of preparing a "Biographical Chart" designed to present comprehensively the names of those most eminent in any department of the world's effort, some knowledge of whom would be included in a scheme of general culture. The arrangement of the chart gave a clear notion also of the prevailing lines of industry at different periods. It was finished in 1888.

In 1889 he published, in the *School of Mines Quarterly*, under the title "Chronology of the Human Period," an interesting chart exhibiting the principal scales used in the division of prehistoric time in their chronological inter-relation.

His book, "Dynamics of the Sun," was published in 1891. Its importance was recognized by eminent scientists.

In the same year, with John B. Farrington, he secured a patent on an electric motor for statical electricity adapted to the use of students, and to experimental purposes.

In 1892 he perfected his invention of a life-saving kite. The J. W. Davis Foldable Steerable Kite was patented in 1893. Various successful tests of its assumed ability to carry a hawser weighing a ton from a ship to land more than a mile away, in a brisk wind such as blows on shore when vessels are in danger of shipwreck, were made from islands in the East River, across Kill von Kull, and from *Brenton Reef*

Lightship to the shore. The trials attracted the attention of newspapers and scientific journals throughout the United States. They were also noticed in English, French and Canadian papers.

In 1894 articles by Mr. Davis on "The Kite as a Life-Saver at Sea," and "Some Experiments with Kites," appeared in *The Engineering Magazine* and in *Aeronautics*.

In 1893 he patented an apparatus for lighting buildings. Models and practical tests promised to be so expensive that Mr. Davis was compelled to defer them. On the completion of other plans, it was his intention to return to this invention, in the practical outcome and high utility of which he had great faith.

In 1895 Mr. Davis secured a copyright on "How to Pronounce the Names that are Liable to be Mispronounced of People and Streets in the Principal Cities of the United States."

The article, "How Translatory Movement may be derived from Vibratory Movement," appeared in the *School of Mines Quarterly* in 1895. A review of "Gillespie's Surveying" was published there in 1896.

Mr. Davis now became anxious to devote more time than the conduct of his school would allow to original research in his chosen lines of mathematical physics and astro-physics; therefore, in 1897 he withdrew from the school and engaged in the solution of certain problems.

In Boston in 1898 he read a paper, "Behavior of the Atmospheres of Gas and Vapor-Generating Globes in Celestial Space," before a meeting of the American Association for the Advancement of Science. An abstract was published in the *Proceedings* of the Association for that year.

In 1899 "Some Properties of a Gaseous Sun" appeared in the *School of Mines Quarterly*.

In 1901 the *Astronomical Journal* contained his article on "The Eruptive Energy of the Stars," which explained the generation of stellar atmospheres by subsidence of globes composed of a mixture of gases the molecules of which are equal in thermal capacity, but unequal in mass.

In the same year, in *The American Journal of Science*, was published "The Motion of Compressible Fluids," which set forth his discovery that equations of motion of compressible fluids have always two solutions representing two real and different motions of the fluid.

Mr. Davis died on November 7th, 1902, leaving a paper, nearly finished, in which are given the exact solutions of the equations for tidal waves, solitary waves, and sound waves.

Mr. Davis was elected an Associate of the American Society of Civil Engineers on June 3d, 1885. He was also a member of the American Association for the Advancement of Science, the New York Academy of Sciences, the New York Mathematical Society, and others.

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